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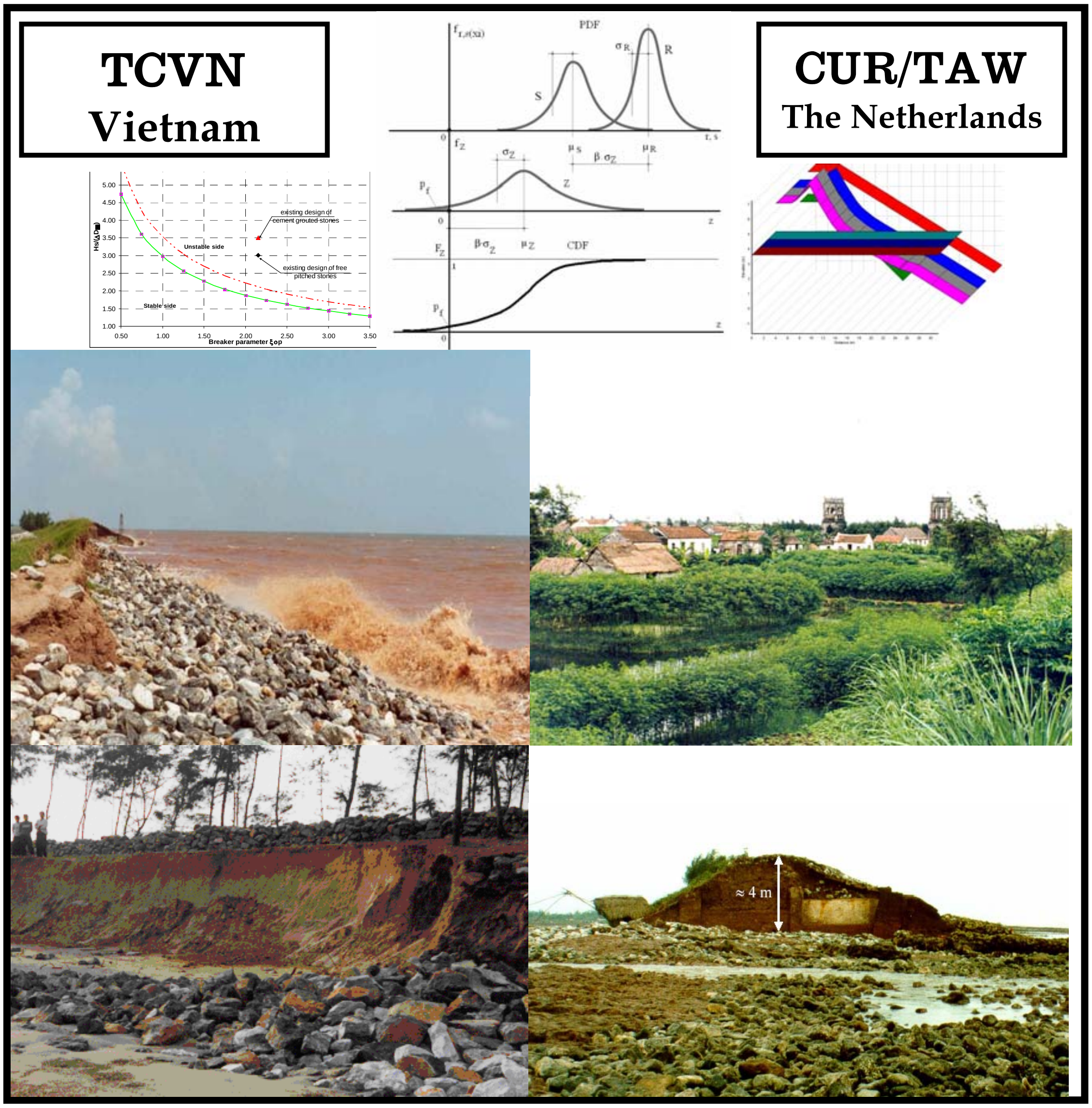

Safety Assessment of Sea Dikes in Vietnam
A CASE STUDY IN NAMDINH PROVINCE

Mai Van Cong

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Master of Science Thesis

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Abstract

Vietnam has about 3260 km of coastline, primarily consisting of low-lying coastal areas which are protected by sea dikes, natural dunes and mountains. More than 165 km of coastline lies within the Red River Delta, a densely populated region which experiences substantial dynamic changes and destruction due to frequent intense impacts from the sea (typhoons, changes in sea level, currents, etc). This dynamic coastline is mainly protected by sea dike system which has been developed for almost hundred years.

The NamDinh Province constitutes part of this coastline, with total length of about 70 km, which is protected by sea dikes. The sea dike system has been heavily damaged. There were many times of dike breach which caused serious flooding and losses. The situation of NamDinh sea dikes can be considered a representative for coastal area in Northern part of Vietnam.

In recent years there has been a number of studies aiming at understanding the situation of sea defences system in NamDinh, assess the safety of the and find the solutions to mitigate these losses for this region. However, due to the lack of data and design tools the results of these studies, somehow, are still limited and the problem is still poorly understood. Therefore adjustment of safety of the existing Namdinh sea defences system is necessary.

This study is initiated with the main focus on analysis and assessment of safety of Namdinh sea dikes. Firstly, the historical development of sea dike system in Namdinh province is analysed base on historical record and collected data. Based on that the possible causes of old-dike failures are carried out. Secondly, the study investigates all possible failure mechanisms and their causes of the existing dikes. Follows by, the safety assessment of the dikes is performed for possible failure modes in term of hydraulic, structural and geotechnical related problems. Finally, conclusions on safety of Namdinh sea dikes are stated and some recommendations (guidelines) of new sea dike design in Namdinh and in Vietnam will be carried out.

The study is based on deterministic and probabilistic approaches. The latest Vietnamese codes and Dutch codes for design of sea dikes and revetments are the basic references for these analyses. Comparisons will be made to applying different design codes for design of sea dikes in Namdinh as well as in Vietnam.

In general, analytical methods are applied in this study. However for solving some specific related problems the advanced mathematic models are also applied as calculation tools such as CRESS and BREAKWAT for some hydraulic related problems; GEO-Slope and PLAXIS for geotechnical related ones; VaP and MathLab models for probabilistic calculations. By doing this study the necessary engineering knowledge and study skill to solve a problem in practice are also achieved.

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CHAPTER 1 INTRODUCTION

1.1 Background

Vietnam is situated in the tropical monsoon area of the South East Asia and is a typhoon prone country. A large number of populations involved mainly in agricultural and fishery sectors is situated in the low lying river flood plains, deltas and coastal margins. Also, there are the important ports and harbours, which are located along the coast. In the other side these areas are the most important potential disaster areas facing Vietnam. Typhoons from the South China Sea bring torrential rainfall and high winds to the coast and further inland. On average four to six typhoons attack the coast annually. Further, the monsoon season coincides with the typhoon season resulting annually in heavy damage, loss of life, and destruction of infrastructure facilities and services. One reason that water disasters are so serious is that most of the population lives in areas susceptible to flooding. The main population centres and intensively cultivated lands in the Red river and Mekong Deltas and the narrow connecting coastal strip of the country are particularly vulnerable to flooding from monsoon rains and typhoon storms. Thus flooding is the most important potential disaster facing Vietnam.

The overtopping of the sea defences causes salt intrusion, which decreases the agricultural productivity. Further the constant risk of flooding discourages farmers to adopt new technology or to invest in other income-generating activities.

The Red River Delta in Northern part of Vietnam is characterized as low lying with an enormous network of river branches with a long line of dikes and sea defences. Most of the sea dikes are built over the centuries mostly due to local initiatives. The sea dikes have generally an inadequate design and are poorly constructed. Due to the bad state of the dikes a significant part of the yearly funds has to be allocated to repairs and maintenance. The length of the coastline is approximately 165 km as the crow flies. In this area, the seashore is often subject to frequent intensity impact from the river (floods) and the sea (typhoon, changes in sea level, current, etc.).

The NamDinh Province constitutes part of this coastline with the total length of about 70 km which is suffering from severe erosion and serious damages of defences system, which can be considered as the representative for coastal problems in Northern part of Vietnam. The defensive measures are mainly consisting of sea dikes and revetments for slope protection. In general, since the coastal erosion and damages of coastal defences occur it results in serious economic consequences as well as social consequences of the concerned locations.

Although, there have been a numbers of reports on the safety assessment of the coastal defences system every year before flood season but these reports were done based only on the experiences on management of the monitors and what already happened of the sea defences system in the previous years. Consequently the risk of the damages is still going on at the high rate and frequently. Therefore, the evaluations of safety of the existing defensive system and analysis of present situation based on the latest design codes are necessary. As the result, some guidelines for new design will be carried out which can be applied for Namdinh sea dikes more accurately.

Thus, in appreciation of the above, the study is initiated with the main focus on evaluations of safety of sea dikes and revetments in Namdinh coastal areas. The latest

Vietnamese codes and Dutch codes for design of sea dikes and revetment will be the basic reference for analysis. Then some conclusion will be pointed out by comparison of applying the different codes for design Namdinh sea dikes and revetments. More over the study also integrates available design methods in order to increase the accuracy and the range of applicability of design tools for similar problems.

1.2 Problem definitions

The main problems in project areas are serious erosion of the coastline and heavy damages of defensive system. The failure of the sea dikes and revetment was caused by the actions of strong storm surges and typhoons while their design parameters were not sufficient. Moreover due to the action of waves and currents the foreshore erosion has occurred seriously which leads to the dikes and the revetments. The specific problems can be listed as following:

- Severe erosion takes place along the coastline of the research area, including the structural erosion and foreshore erosion. The structural erosion rate is about from 10m to 20m per year while the foreshore erosion causes loss of 0.3 to 0.6 m thickness of sand in front of the dikes system. This leads to fast retreat of coastline if there are not sufficient and in-time counter measures.

- Beach erosion, dike breach due to typhoons, storm surges, and wave actions caused retreat of up to 3000m of the shoreline during the last 100 years. Total area of land loss is approximately 15,000 ha (nearly as big as the current area of the HaiHau district).

- Strong storms with wind-strength of 9 to 12 Beaufort cause houses to collapse, killing people and huge property loss. In the last period of 20 years from 1976 to 1995, storms took away 4,028 houses, 6 fishing ships sank, and 25 people died and 34 people were injured.

- Dike breach: seawater overflow into to the hinterland resulted in flooding and salt intrusion in cultivated land. Practical statistics showed that 38,273 ha cultivated land was impacted by salt, and 76,474 tons of food was lost. Salt mining fields, and shrimp hatching ponds were also heavily damaged.

- Storms surge often accompanied with high tides caused damage of Namdinh sea dikes almost every year. During the period from 1976 to 1995 about 934,000m3 of earth and 30,400 m3 of stone were taken away from the sea dikes. Therefore the expenditure on maintenance is very large (in order of millions of Euro).

- Heavy damages and collapses of the defensive system, especially the dike system and revetments. Many sections of dikes and revetments failed and breached induced by variety of failure modes. This caused flooding in the wide area along Namdinh coastline and as the consequence, it leaded to loss of land, economic archives and even loss human’s life

- The sea dikes system in Namdinh has 2 main functions of flood defence and protection of inland from erosion. The reason is evident because these dikes exist already for more than 1000 years. This means that the dikes must be there in any cases. However, nearly all the dikes which were constructed in the past were designed by very old method and only based on the experiences of the local people. For the time being, the dikes system seems to be insufficient respect to the actual boundary conditions.

It is apparent that the coastline erosion and the damages of defensive system lead to many effects on the social and economic development in the area. In response the central and local authorities have undertaken some efforts in order to restrain the possible adverse consequences and as future defensive measures, some sections of new sea dikes had been built. However, due to budget constrains, the lack of suitable design methodology as well as strategic and long-term solutions, such efforts still remain limited to reactive and temporary measures. Following Figures are showing the recent photos at HaiHau coast. The photos show some impressions view about the problems and how serious it is.

Figure 1.1: A damaged dike section

Figure 1.2: HaiTrieu Village in 1995

Figure 1.3: Abandoned HaiTrieu in 2001

1.3 Scope of study

The scope of this study includes two main aspects:

- Deterministic assessment of safety of the sea dike system in Namdinh province by applying the sea dike Design Standards of Vietnam and the Netherlands. The safety assessment will be done by investigation of all possible failure modes and their mechanisms, which may occur at Namdinh sea dikes. The investigation of the possible failure modes will be performed by taking in to account the hydraulic, geotechnical, and structural aspects.

- Brief study on probabilistic approach for investigation of safety of the dikes system. In this study the level II of probabilistic calculation is applied for safety assessment of one representative cross section of the dikes.

1.4 Aims of study

The aim of this study can be outlined as follows:

- To understand the problem by analysis of the possible failure mechanisms of sea dikes and revetments along NamDinh coastline. The analyses of original situation to current situation are based on collected data and site visit.
- To compile an overview of all relevant potential failure mechanisms, covering hydraulic, geotechnical and structural aspects.
- To identify the failure mechanism probabilities to be quantified with priority.
- To review the design methodology which was applied for existing sea dikes and revetments in NamDinh
- To compare of the safety of Namdinh sea dikes by applying Vietnamese Design Code and Dutch Design Code of sea dikes and revetments.
- Deriving conclusions by comparison of applying Vietnamese Code and Dutch Code for design of sea dikes.
- To integrate available design methods of sea dikes by applying the probabilistic design.
- To increase the accuracy and the range of applicability of design tools for sea dike design in Vietnam.

1.5 Study approach

- Collect necessary data from all possible sources covering the topic.
- Point out the future predictions of the failure mechanism probabilities for Namdinh sea dikes based on the analysis of the historical failures of the dikes.
- Review previous related studies which deal with Namdinh coastline.
- Review the existing dike design of sea dikes in Vietnam.
- Deterministic assessment of the safety of Namdinh sea dikes by applying Vietnamese and Dutch codes. Includes:
    1. Hydraulic related problems.

2. Geotechnical related problems.
3. Structural related problems.

In this section, using the numerical models for calculations of some specific problems is necessary. The models, which will be used, are as following:

i- CRESS and BREAKWAT programs: for calculations of some hydraulic related problems.

ii- GEO-SLOPE (Canada) and PLAXIS (The Netherlands) for computation of geotechnical related problems.

- Analyze the differences of results by applying the different codes. Base on that to find out the remarks for new design of sea dikes along Namdinh coastline and in Vietnam.
- Probabilistic assessment of the safety of the dikes.

1.6 Outline of study

- The general information of the study is given in chapter 1
- In chapter 2, description of study area and boundary conditions including the natural and existing conditions are given.
- The study of historical record and review of previous related studies are presented in chapter 3. In addition to that the review of design consideration of sea dikes is given. This will be treated as literature review.
- In chapter 4, there will be investigated all kind of failure modes which may occur with Namdinh sea dikes. Furthermore, the analysis of these failure mechanisms will also be performed.
- Chapter 5 is the main part of the thesis which introduces the safety assessment of sea dikes in Namdinh. The assessments will be carried out by applying Vietnam and Dutch design codes. After that some remarks for new design will be given based on the comparisons between both codes.
- In chapter 6, as an integration of the new design method, the study will carry out an overall safety base on probabilistic assessment of the safety of Namdinh sea dikes.
- Finally, the conclusions and recommendations will be treated in chapter 7.

CHAPTER 2 BOUNDARY CONDITIONS

2.1 Natural condition

2.1.1 General description about study area

The coastal zone of Namdinh is roughly 80,000 hectares in size which is protected by about 70 km of sea dikes. The area is naturally divided into 3 sections by 4 large estuaries: the Ba Lat (Red River), Ha lan (So River – has been cut-off), Lach Giang (Ninh Co River) and Day (Day River), from north to south the sections are[Vu et all] :

- Section 1: from Ba Lat estuary to So estuary belongs to Giao Thuy district, about 27 Km long.
- Section 2: from So estuary to Ninh Co estuary, belongs to HaiHau district, 27 Km Long.
- Section 3: from Ninh Co estuary to Day estuary, belongs to Nghia Hung district, 16 Km long.

The erosion or accretion rates vary depending on the position of the section that faces to the sea or the proximity to the estuary. (See Figure 2.1)

Accretion at the estuaries:

- *Ba Lat estuary* : The accretion at the Ba Lat estuary has been forming for about 30 - 40 years. Firstly this accretion is only one big alluvial ground connected to a section of sea dike belonging to the Giao Thuy district, forcing the Red river to run northward via the Lan mouth to the sea. The accretion ground grew bigger, year after year, then flood flow from the Red River has divided the ground into 3 parts: the inner ground (next to the former sea dike), Con Ngan ground (in the middle), and Con Lu ground on the outer area facing the sea.

- *Day estuary*: Alluvial ground at Day estuary - named Con Xanh ground - belongs to Nghia Hung district. This new delta has been formed by the Day river, the delta is growing very fast, since 1975 the delta has encroached about 8 Km seaward. From 1931 to date there has been 2 series of dikes, which were constructed for land reclaimation, and a new commune (named Nam Dien) was formed with an area of 1,2000 ha.

- *Lach Giang estuary*: this is also an accretion estuary and the delta here is not as big as the other ones mentioned above but this is one of the main national channels connecting the seaway to the inland waterway system. Lots of sand has been dredging in order to maintain the shipping channel.

Erosion situation: At the locations far from the estuary that face the sea the erosion problem is taken place and quite alarming. The erosion is happening along the coastline from the southern coastline of Giao Thuy district to the coastline belonging to the HaiHau district and also taking part of northern coastline of the Nghia Hung district. At the erosion locations the beach width is very narrow, only 100 - 200m at the low tide. According to the records of the local Dike department in Namdinh, the averaged yearly retreat speed during the period of from ***1900 to 1954*** was about ***35m to 50m*** while from ***1954 to 1973*** was about ***15m to 25m*** and in period of ***1973 to 1990*** was ***8m to 10m***.

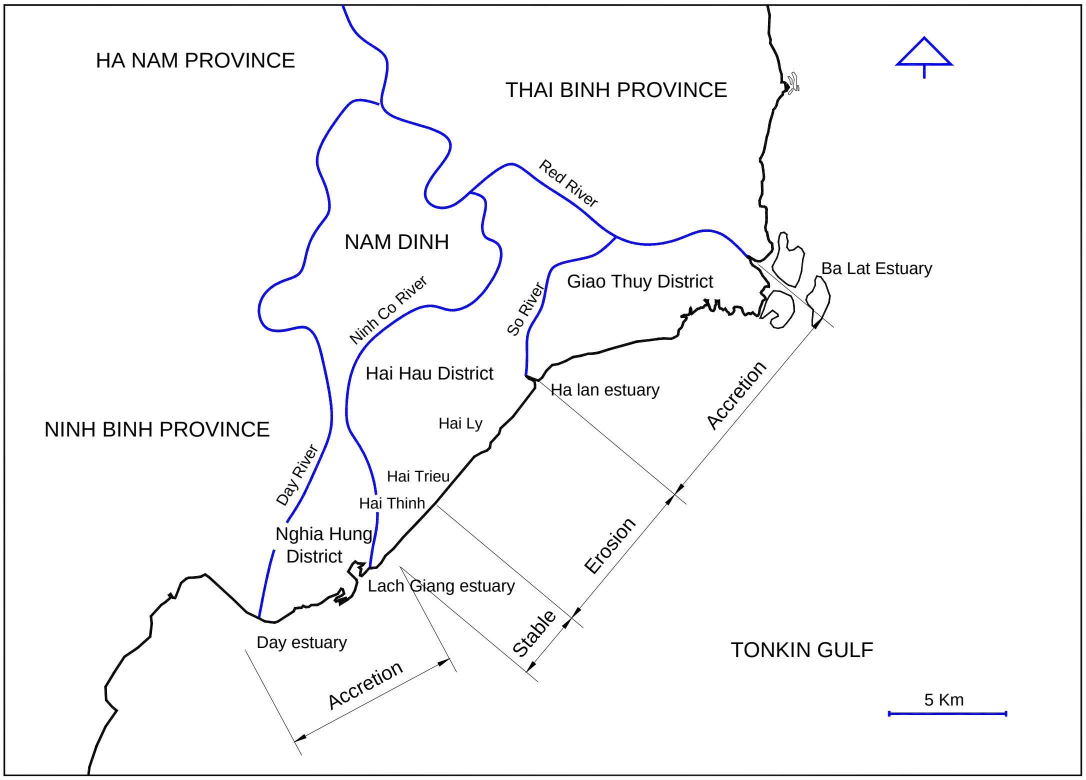


Figure 2.1: The current situation of Namdinh coastlines.

2.1.2 Delta topography

According to Le, Ngoc Le, (1997), the delta has flat topography, gradually sloping from northwest to southeast with an altitude vary from 10-15m to mean sea level over a distance of 150 Km. During the mid and late Holocence period, the mountainous bottom of the Tonkin Gulf filled up with alluvium. In the middle of the delta, mountains and hills can be found, linked to the geological formation under the alluvial sequences. The delta can be subdivided to three parts: (1) the Rim Plain, (2) the Central Plain, (3) and the Coastal Plain. The Rim Plain was not submerged in the mid-Holocene period and it is covered with ancient alluvium and dotted with sparse hills and mountains, which form part of underlying geological foundation. The area is elevated 3 m above mean sea level. The Central Plain is the area built with new alluvial from the Red River and the Thai Binh River and it was submerged in the mid-Holocene period and has been impacted by both rivers and the sea (Le, Ngoc, Le, 1997). The area elevates 1-3m above mean sea level and its topography is one of low-lying lands with mountains and hills. The Coastal Plain consists of young alluvial deposits. The topography is flat, varying from 1m below mean sea level to 1 m above mean sea level with the presence of beach ridges. The pro-delta zone (the most seaward portion of the sub aqueous delta) has a depth of 20-30m covered with silt and red silty clay (Hoi and Tuan, 1994).

Upstream, in the mountainous area surrounding the delta, the Red River is confined to a straight narrow northwest-southeast aligned valley (Figure 2.2), produced by the Red River Graben (a sunken area between two roughly parallel faults, the faults converge toward one another below the surface, so that they look like the letter “V” in cross section). This major tectonic structure can also be traced south-eastwards deep beneath the Quaternary sediments of the delta plain and into the Tonkin Gulf. It acts as a major sediment trap (Fontaine and Workman, 1978).

Recent studies about geology and geomorphology of the Red River Delta have confirmed that there's no relation between the tectonic activities and the erosion problem at coastline of Namdinh.

2.1.3 Soil characteristics and Geological features

Namdinh province has been formed by the rivers in Red River system, soil in Namdinh has alluvial characteristics. Outside the sea dike, the coastline has been shaving due to action of waves and tide current, the erosion is taking away the small grains causing the coarsening of the grain size of the beach.

According to the geology investigation document of the Hydraulics Engineering Survey and Design Service of Namdinh, strata structure of Namdinh coast has 3 following layers:

- The upper layer is sand, covering all over the beach with a thickness range from 0.5m to 2.0m. Grain size ranges from 0.1mm to 0.15mm.
- Under the upper layer is a clay layer with thickness ranging from 0.5m to 1m. This is the original clay layer of the beach, in plastically flabby state.
- The third layer is a coarse sand layer with a thickness of more than 5m.

With this structure of the strata we can easily realise that Namdinh has a vulnerable beach. If the upper layer is washed away the stability of the dike will be seriously threatened.

2.1.4 Sediment transport conditions

The shoreline of Namdinh is in opening sea, not protected by islands or large tidal barriers. The sediment supplied by rivers is accumulated in the near shore zone close to the river mouth and is not transported along the shore in any significant amounts. Therefore, sections of the beach situated relatively far from the river mouth in the range of ten kilometres are not nourished by river sediment.

The beach slope is rather gentle with average value that fluctuates from 1:150 to 1:300 along the coast. But near the dike in a distance of about 300m seaward from the dike toe, the beach is relatively steeper; the slope here varies from 1:50 to 1:100.

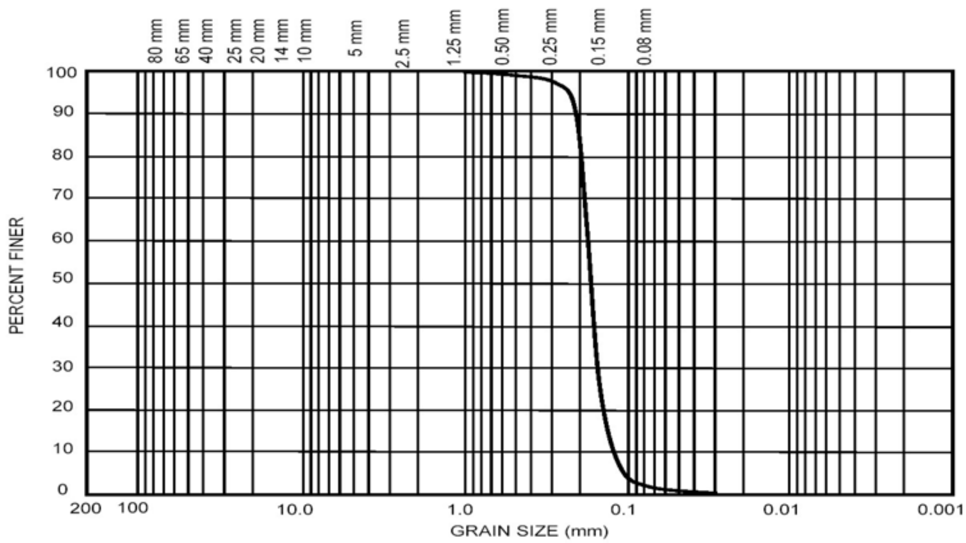
Table 2.1: Sediment load composition on the shoreline **[Pruszak et al. 2001]**

	Sand	Aleurite	Clay
Percentage	22%	64%	14%

Figure 2.2 illustrated a different approach to particle size distribution on the coast as referred to Hung et al. (2001).

A rough assessment of longshore sediment transport in the coastal area of the Red River estuary indicated that the total annual longshore sediment transport is about 5% of the whole annual Red River sediment discharge that remains in the near shore zone, Pruszak et al. (2001). During the winter monsoon the longshore sediment transport is directed southwest. In the summer period it reverses to the northeast. A general scheme of sediment flux showing the rate of sediment discharge to the sea by the main Red

River branches together with the division of the coastline area into three parts is presented in attached Figure 2.3.



Test	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
1	0.0	0.0	98.0	2.0	

LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_C	C_{LF}
		0.199	0.167	0.157	0.138	0.121	0.113	1.01	1.5

[Source: Sea Dyke service Department, Dec. 2001]

Figure. 2.2: Sieve curve of beach material in HaiHau coast

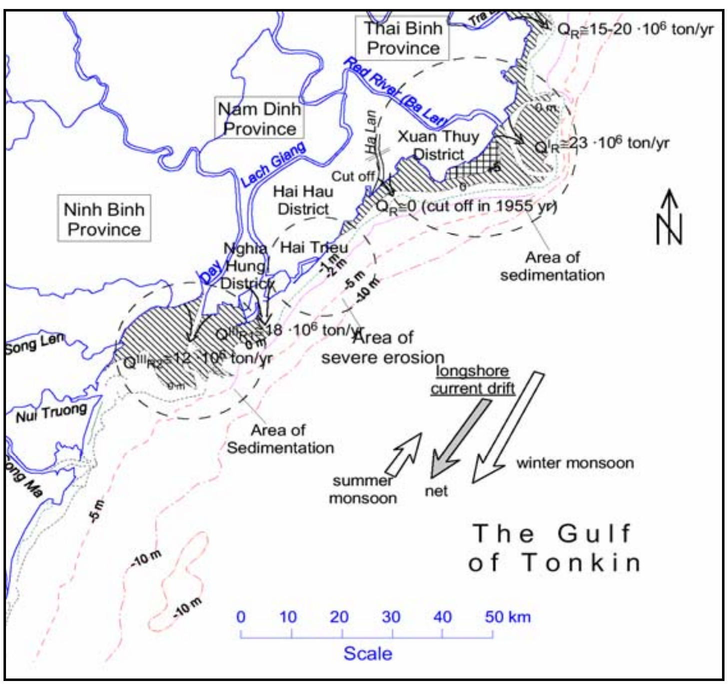


Figure. 2.3: Local sediment budget at Namdinh coast (Pruszak et al. 2001)

2.1.5 Climate and Meteorology

Namdinh is situated in tropical climate area with a pronounced maritime influence. The average annual rainfall is 1600 to 1800 mm, 85% of which occurs during the rainy season (April to October). The heaviest rainfall occurs in August and September, causing intensive flooding in the delta due to overflow of the riverbanks.

The winter is cool and dry, with mean monthly temperatures varying from 16^{o}C to 21^{o}C. Fine drizzle is frequent in early spring, after which the temperatures rise rapidly to a maximum of 40^{o}C in May. The summer is warm and humid, with average temperatures varying from 27^{o}C to 29^{o}C. The prevailing winds are Northeast in the winter, and South and Southeast in the summer.

Typhoons and tropical storms are frequent between July and October. During the period from 1911 to 1965 the region withstood 40 typhoons. However, the frequency of storms and typhoons appears to have increased in recent years. Typhoon storms usually come from the west pacific, through the Philippines or Eastern Sea. They then shoot into the coastal areas of South China and Vietnam. Among the typhoons that occurred from 1954 to 1990, strong winds with grade 12 were observed for 31 cases. The annual average number of typhoons is about 5, but more than 10 were observed in 1964, 1973 and 1989. The severe latest typhoon hitting Namdinh province was Nikki in 1996, causing a surge of 3.11m at the HaiHau district coastal area.

Typhoons also bring about periods with heavy rains, (over 100 mm/day, possibly 300-400mm/day) causing severe flooding. The rains, which affect areas in radius of 200 – 300 km, may become terrible natural calamities. When such storms break over the main land, a huge amount of water is released, damaging the sea dikes (rainfall erosion), and flooding the coastal areas.

2.1.6 Oceanography

2.1.6.1 Tides and tidal currents.

According to tidal map of Vietnam, Tide at Namdinh is diurnal with tidal ranges varying from 3 - 4m. The records at VanLy gauging station show that tide and water level at VanLy is similar to Hon Dau gauging station. The tidal Table of the General Department of Hydrometeorology reveals that the water level at VanLy station can be deduced from the data at Hon Dau station with coefficient of 0.95.

Observation at Hon Dau station shows that tide in this area is purely diurnal there is one spring tide and one neap tide every month (period more or less 25 days) and one high tide and one low tide a day. Tidal range in is about 3.0m in the spring tide.

Table 2.3: Extreme tidal water level in period of 19 years at Namdinh coast

No.	*Location*	*MSL (cm CD)*	*Max. HW (cm CD)*	*Min. LW (cm CD)*	*Tidal range (cm)*
1	Ba Lat	185.60	346	-7	353
2	Ha Lan	185.30	345	-7	352
3	VanLy	185.00	344	-7	351
4	Lach Giang	185.00	345	-8	351

[Source: Vietnamese Water Resources Institute, 2002]

According to the tidal model of the Vietnamese Hydraulic Institute the tidal current at Namdinh is irregular diurnal. The diurnal character of the tidal current decreases southward, even at Lach Giang estuary the tidal current already is irregular semi-diurnal. This means that the variation of tidal level does not coincide with the tidal current.

Table 2.4: Extreme tidal current in period of 19 years at Namdinh coast

No.	*Location*	*Flood tide*		*Ebb tide*	
		Velocity (Cm/s)	*Direction (Dgr. N)*	*Velocity (Cm/s)*	*Direction (Dgr. N)*
1.	Off shore Ba Lat	59	348	57	174
2.	Off shore VanLy	45	310	37	159
3.	Off shore Lach Giang	26	355	41	145

[Source: Vietnamese Water Resources Institute, 2002]

According to field observations done by Hung et al. (2001), wave-induced longshore currents have average value of 0.2 to 0.4 m/s and maximum of 0.7 to 1.0 m/s at depth of 2.5m. These Figures include the tide current velocity (Hung et al. 2001). Longshore wave-driven currents are south-westward in the winter and north-eastward in summer.

According to the Vietnamese Hydraulic Institute, a current at the Namdinh coast always exists due to winds, this current flowing in direction northeast to southwest. The current is stronger in the winter time (November to March), and he average wind current in winter is about 30 cm/s to 40 m/s, while in summer it is only 10 to 20 cm/s.

2.1.6.2 Wind

Since there is no offshore island, and it has relatively flat and low-lying topography, HaiHau is an area exposed directly to the open sea, the area is subject to the winds generated from every direction. In the winter time (from October to March) the dominant wind directions are north, northeast and east. In summer (from May to August) the dominant wind directions are south, southeast and southwest. April and September are considered to be transition times.

In this study the observed wind data at Bach Long Vy Island was used (Tonkin Gulf, 20.133^o latitude; 107.72^o longitude).

Figure.2.4: Main seasonal wind directions in northern Vietnam

Table 2.5: Wind data at Bach Long Vy Island (observation: 1975 - 1995)

Class (m/s)	*N*	*NE*	*E*	*SE*	*S*	*SW*	*W*	*NW*	*Sum*
1-5	843	3,103	2,843	1,875	1,858	578	277	320	**11,697**
6-10	505	5,160	1,378	810	3,440	530	77	108	**12,008**
11-15	156	2,013	73	79	1,043	65	6	9	**3,444**
16-20	90	863	11	23	77	4	2	19	**1,089**
21-25	16	27	0	2	5	1	0	5	**56**
26-30	3	4	0	1	2	3	0	3	**16**
31-35	3	1	1	0	4	0	0	0	**9**
36-40	1	0	1	1	0	0	0	1	**4**
Sum	1,617	11,171	4,307	2,791	6,429	1,181	362	465	**28,859**

Storms/Cyclones: As referring to the topographic map, the beach of the study area has a very gentle slope, which creates a relatively wide zone for wave transformation and energy dissipation. Apparently, only monsoon waves, severe storms or typhoons, with high rainfall, extreme wind speed, high wave and storm surges, cause severe threats to the local natural beach and the existing coastal structure.

In the study area, according to the weather observation record, there were about 4 typhoons occurring in a year on average. August and September are the most critical periods to encounter floods and storms. In August and September, storm winds are generated from NE with velocities of 20 m/s, and in some cases even up to 48 m/s. Typhoons are normally accompanied by storm surges. See Table 2.6

Table 2.6: Storm surge at Namdinh coast

Surge level (cm)	0 - 50	50 - 100	100 - 150	150 - 200	200 - 250
Frequency related to number of storms (%)	35	38	17	8	3

[Source: Vietnamese Water Resources Institute]

2.1.6.3 Waves

The sea at NamDinh is open sae (there is no offshore island) so the wind fetch is long enough for wave growth and approaches the shoreline without any obstacles, which can cause considerable damage to shoreline and sea dikes. According to observation in period from 1975 to 1987 waves at Namdinh had following characteristics:

- In winter (from September to March): In the winter, the sea was much more rough sea than in the summer. Wave height is about 0.8m – 1.0m, with periods varying from 7 to10 seconds. Predominant wave direction was northeast, and makes angles of about 30^o to 45^o with the shoreline.
- In the summer (from April to August): In the summer there are less rough sea days but strong storms usually happen in this season causing severe damage to the dike system. Average wave height varies from 0.65m to 1.0m with period ranging from 5 to 7 seconds. The prevailing wave direction is south and southeast.

2.2 Present situations of sea dike system.

2.2.1 Sea defence system in NamDinh province

Sea dikes play a dominating and important role concerning shoreline defence structures in Vietnam, and for Namdinh province, the dike systems are totally prevailing. The defence strategies are regarding to construction, maintenance and rehabilitation which is overall governed by the Ministry of Agricultural and Rural Development (MARD) but is operationally run by the Department of Dike Management and Flood Control (DDMFC), which handles more than 3,000 km of coastal and estuarine dikes (Pilarczyk and Vinh, 1999). The main objective for DDMFC is to secure communities in coastal areas from erosion and flooding and thus increase agricultural production and income.

Construction of new dike systems and upgrading of old ones is a continuous process. In VanLy, for example, the average annual coastline retreat has resulted in one destroyed dike line every 10 years. Due to the lack of proper equipment, upgrading and repair (in case of breach) of the front dikes are rarely possible and the land behind the dike is lost to the sea. Dike maintenance costs are extensive and in Namdinh they represent nearly 95 percent of the total sea defence budget (VCZVA, 1996).

The normal design wave height is based on an annual frequency of exceedance of 5 percent of time, which is determined by both investment costs and levels of protection. The dikes are fundamentally constructed to withstand concurrent design events, which are reflected in the employed dike crest elevation formula given by $z_{crest} = z_{tide} + z_{storm\ surge} + z_{wave\ run\text{-}up} + z_{free\ board}$, where z is elevation and the subscripts are self-explanatory. However, funding problems and shortage of equipment for example vehicles have affected the construction of the dikes and thus resulted in both weak structures and serious overtopping (salinity intrusion). In the future the economical development in the coastal zone will expand and thus it is expected that investments will increase and more money will be put into erosion control, i.e. better defence systems. The Vietnamese design standards are somewhat out of date and must be revised in order to meet contemporary international knowledge (Pilarczyk and Vinh, 1999).

The dike system at Namdinh is characteristically positioned as shown in Figure 2.5 and Figure 2.6. When a breach takes place, the section dikes help to limit flooding and the second dike will be the new first line of defence. In general, the second dike is mainly made of soil (no proper revetment) and thus it is weaker than the first. However, these dikes must and will be reinforced when the water reaches them; otherwise they will not long lasting. The distance between the dikes varies roughly 200 meters. The land areas between the dikes are also divided into sections varying between several hundred meters up to 3 km. The division into sections causes only limited areas to be flooded when a breach occurs at the front dike and without sections greater land areas would have been destroyed at once. Recent photos of the front dike reveal major erosion problems and clearly show the earth core of the dike as seen in Figure 2.7. The photo also illustrates the casuarinas tree, which is frequently planted and used to reduce wind speed and bind the shoreline soil. The tree is common not only at Namdinh coast but also along Vietnam coast in general.

According to the VCZVA (1996), the front slope of the dikes in NamDinh province is normally 1:3 to 1:4 and the crest elevation lies around 5 to 5.5 meters above mean sea level (MSL). The earth core consists of material from local sand and clay resources, which strongly affects the durability of the dikes since the fine soil is easily flushed out to sea. On top of outer slope the revetments were constructed of natural stones and/or artificial blocks on a layer of clay. A characteristic dike cross-section is shown in Figure 2.8. In total, dikes protect 95 % of Namdinh coastline.

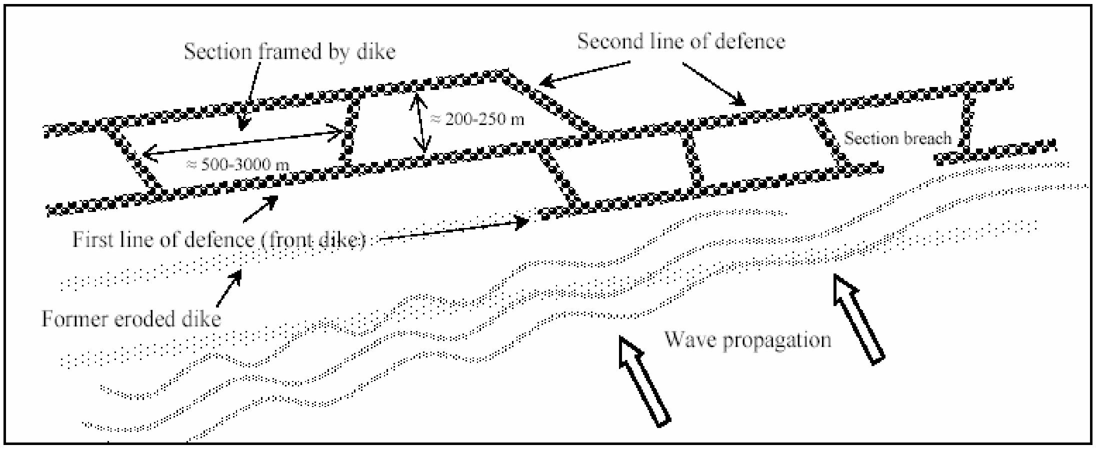


Figure. 2.5: Sketch of double dike system at HaiHau beach

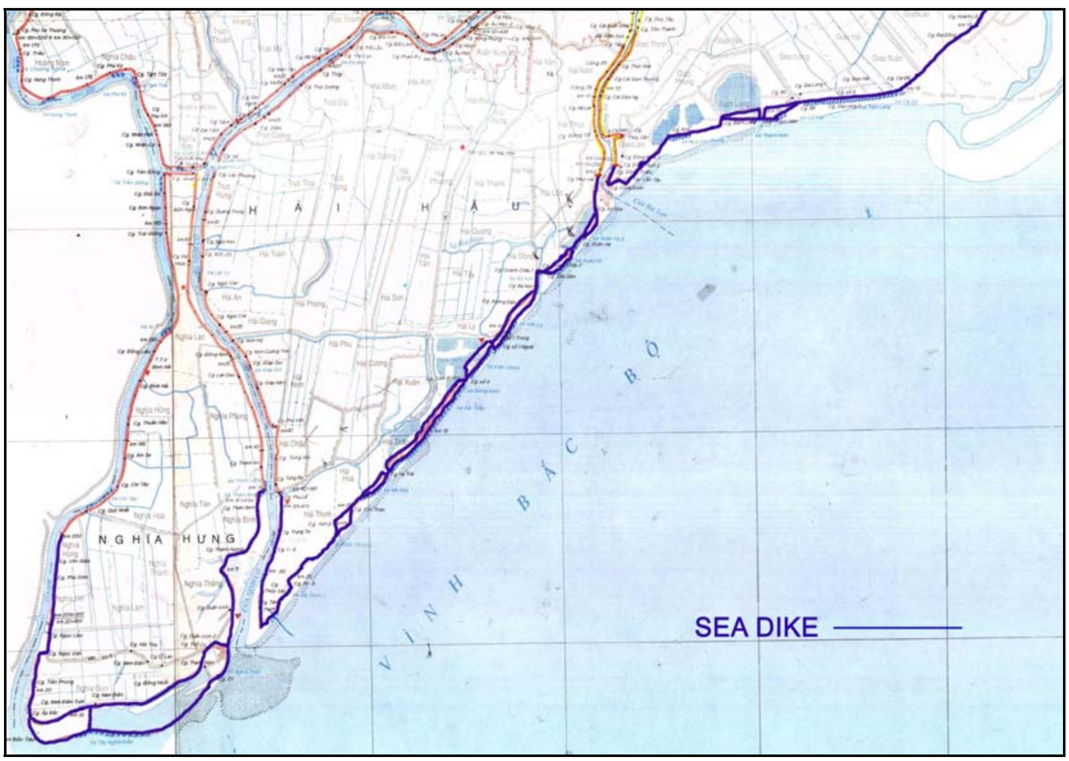


Figure. 2.6: Sea dike system in Namdinh province

Figure 2.7: Severely eroded dike with planted casuarinas trees at HaiHau beach.

Figure.2.8: Characteristic cross-section of an eroded dike near VanLy village

2.2.2 The current situation of sea dikes in Namdinh province.

The cross section of the dikes in the study area is mostly the same for all the section along the coast. It can be described as the representative design cross section and shown on Figure 2.9.

Generally, the length of coastline is about 70 km which passing three coastal districts (with length of sea dikes): Xuan Thuy (32 km), HaiHau (33 km) and Nghia Hung (26 km). The dikes have been improved under WFP 15 km, divided in different sections, various located to the direction of wave attack.

Design of cross section :

- Design tidal water level MSL + 2.29 m (probability of 5%) storm surge from calculations by formula and observations (+ 1.0 m) design water level MSL + 3.29 m
- Crest freeboard 0.21 m
- Calculated crest height MSL + 5.50 m (slope 1:4)
- Dike profile: seaside slope 1:4; landside slope 1:2; crestwidth 4 m

The sea slopes are protected by pitched stone revetment:

- Below MSL + 3.5 m the thickness of 45 cm (calculated formula for rock revetment); block dimensions 0.50 x 0.50 x 0.45, the average weight is approximately 250 kg
- Above MSL + 3.5 m the thickness is of 0.30 m
- Layer of gravel has the thickness of 25 and 15 cm, and layer of loamy soil is 70 and 50 cm.

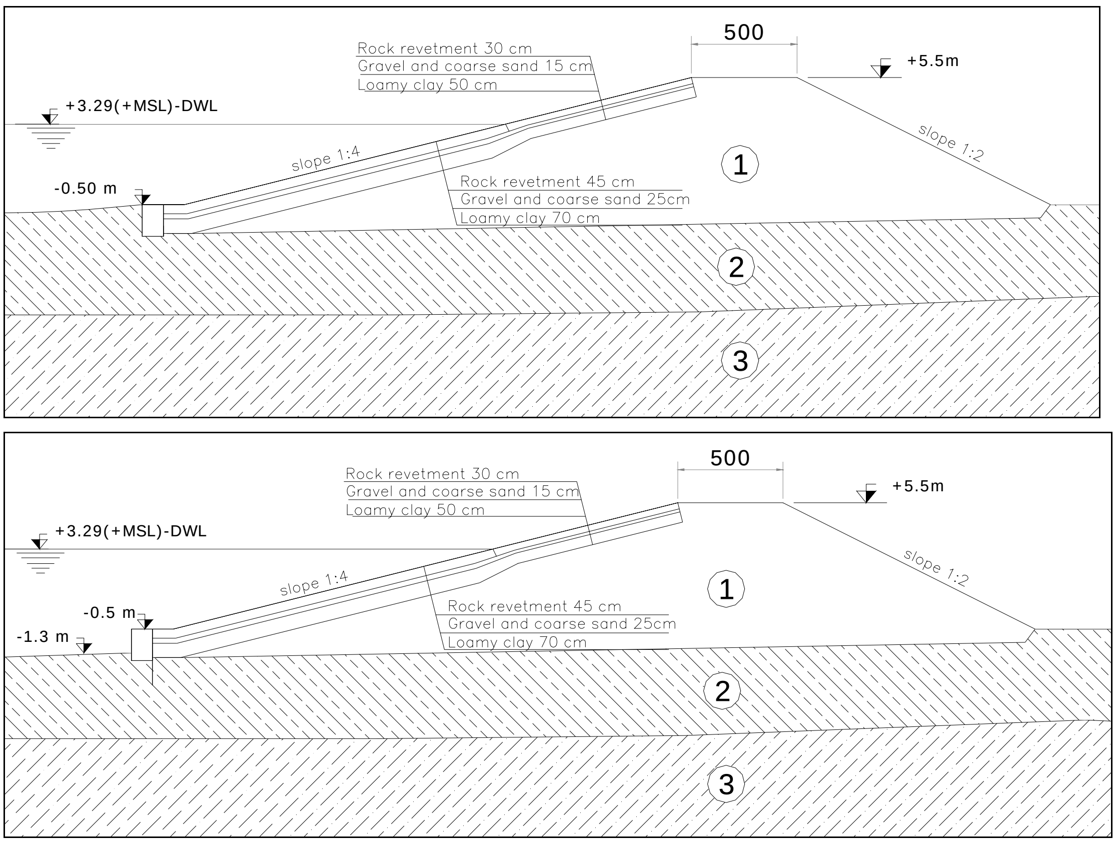


Figure 2.9 Representative cross section of sea dikes in Namdinh

CHAPTER 3 OVERVIEW OF PREVIOUS STUDIES AND REVIEW OF DESIGN CONSIDERATION FOR SEA DIKE

3.1 Overview of previous studies.

The aim of this chapter is to give an overview of the historical changes of coastline and historical development of the dikes system along Namdinh coasts based on the collected information and review of previous studies. Moreover the consideration for design of sea dike is also given as a general overview at the end of this section.

3.1.1 Historical changes of Namdinh coast

Thousands of years ago people in Namdinh coastal zone tried to gain land by building dikes around coastal areas. This is known then as “reclamation system”. The system consists of two parallel dikes with a distance of about 250m in between.

The severe erosion in HaiHau district occurred 100 years ago. Observation has shown that natural climate changes and man-made structures built in the area could be the reason of these processes., the changes of Namdinh coast during last century are shown In the Figure 3.1 and Figure 3.2 briefly. The Figures show that erosion took place in the two southern districts (HaiHau and Nghia Hung) and strong accretion in the northern district (Xuan Thuy) in Namdinh province.

As can be seen from Figure 3.1, the retreat of the shorelines at HaiHau coast for the period 1912-1981 has been estimated approximately 2 km. Thus the erosion rate over these years can be estimated as 24 m per year. A more detailed view of the coastline changes at HaiHau beach is given in Figure 3.2. It should be noted that the coastline in Figure 3.1 and 3.2 is taken from the sea chart map with small scale (order of 1/100,000). These maps are not detailed enough to visualise the sea dike system at the study area. However the analysis can give an overview about the evolution trend of the coastline in the area.

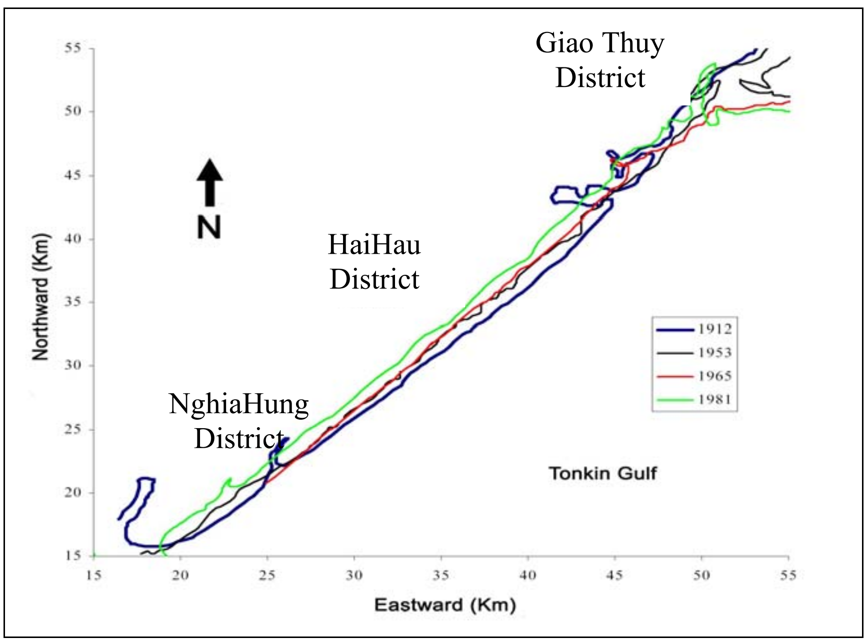


Figure 3.1: Coastline change at Namdinh province from 1912 to 1981

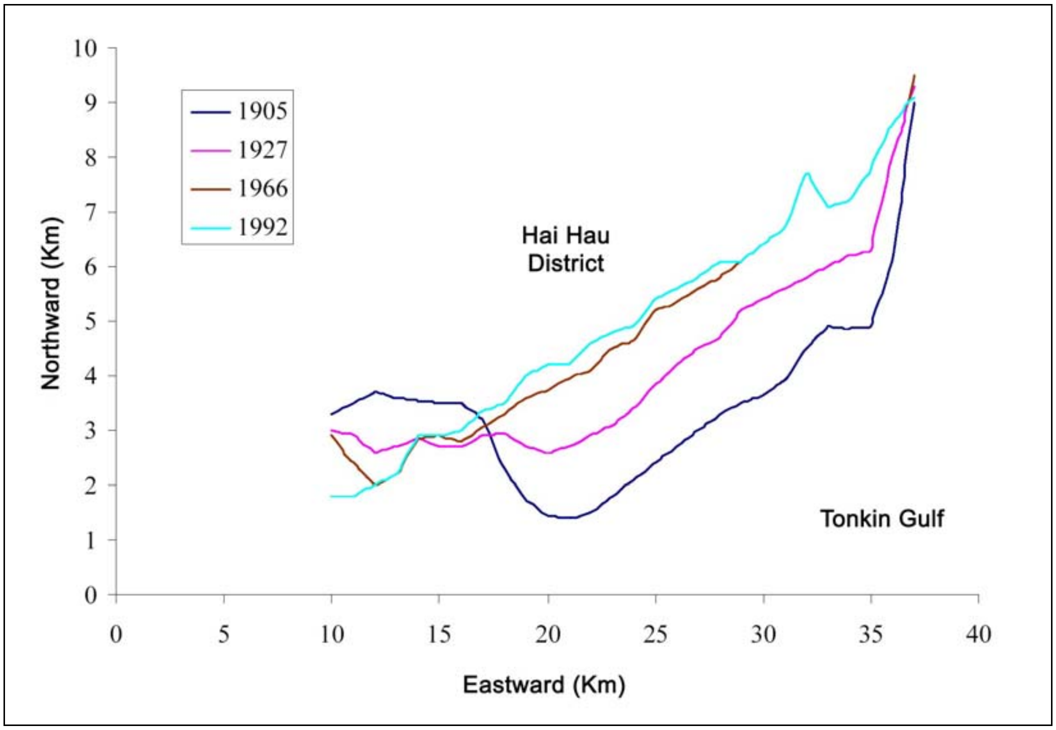


Figure 3.2: Coastline change at HaiHau beach from 1905 to 1992 (Hung et al., 2001)

Figure 3.3 shows the condition of the HaiHau beach in 1995 and 2000 by means of photographs. It is clearly indicated in the photos that the beach has suffered from erosion. The dikes in the area have suffered breach and collapse and currently in poor condition.

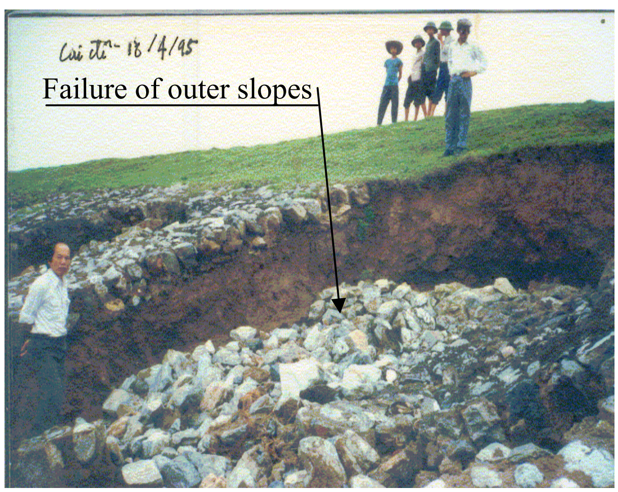


Figure 3.3: A failure of sea dikes at HaiHau in Namdinh(April 1995)

As shown in Table 3.1, the erosion rate was different in each commune. The average erosion rate from HaiDong in the north to HaiThinh in the south was about 16 m/year. The overall trend of erosion rate was decreased to the south. A maximum erosion rate was located in HaiLy (VanLy), which is located in the northern part.

Table 3.1: Summary of erosion rate from 1972-1996

Location	*Erosion distance (m)*	*Erosion rate (m/year)*
HaiDong	600	25
HaiLy	720	30
HaiChinh	360	15
HaiTrieu	300	13
Hai Hoa	180	8
HaiThinh	100	4

3.1.2 Overview of previous studies

Previous study have been undertaken to come across an explanation of the phenomena that occurred in this area over the last few decades. Most of the studies related to morphological analyses, sediment transport computation and shoreline revolution.

i) "Research on Prediction and Prevention of Shoreline Erosion at Northern Part of Vietnam"*, Oceanography Sub-Institute in Hai Phong, Vietnam (2000).*

The study investigated the reasons for erosion at the northern coast of Vietnam – Including Namdinh province. It included the prediction of coastal evolution and recommendation of the shoreline management. Conclusions referring erosion problem in HaiHau district were indicated in this study which can summarised as follows:

- *Indirect reason:* Erosion of HaiHau beach is related to natural evolution of Red River Delta and human activities that can cause reduction of sediment supply to the sea.
- *Direct reasons:*
  + Combination of longshore and cross-shore transport is a direct reason that causes erosion problem in this area.
  + Closing of HaLan estuary (So river) also contributes to the erosion process. Due to this closure, there is some reduction of sediment transport to the sea.

In this report, there are also some general proposed solutions for erosion problems along the beach of Namdinh. It was suggested to build a groin system for HaiHau beach.

ii) " Coastal Processes in the Red River Delta Area, Vietnam" *Pruszak et al. 2001*

The study focused on analysis of the evolution of the Namdinh coastline by means of using morphological model, UNIBEST. The study is concentrated in one segment of coastline that is most vulnerable to destruction, that of the HaiHau beach. For this modelling, 20 year-recorded waves in the depth of 20m is used. Thus, the wave analysis used Krynov spectral method. The result indicated gradually smoothing-out of the shoreline configuration and reduction of the intensity of erosion.

It was concluded that the reason for the erosion in HaiHau area is the changes in climate and complexity of topography that influence the sediment transport surrounding area. However, it is predicted that the erosion would be decreased due to improvement of hydraulic structures along the beach and equilibrium of the coast. See Figure 3.4.

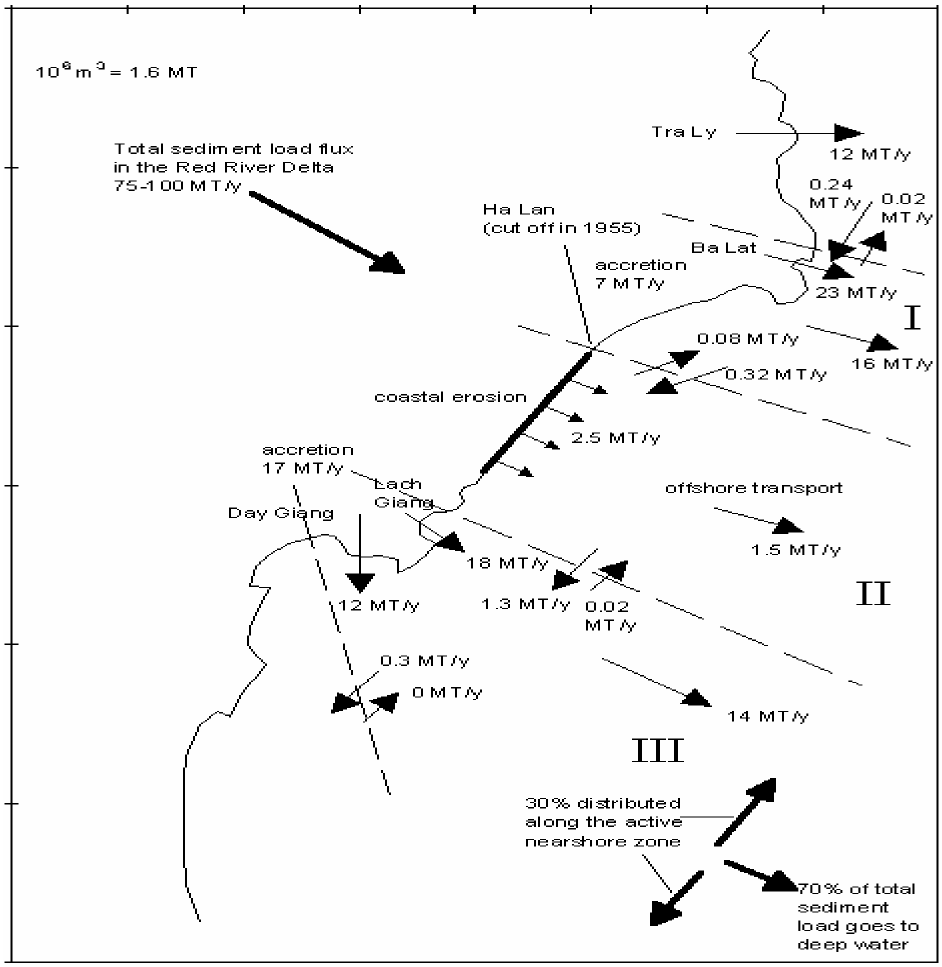


Figure 3.4: Sediment transport along the Namdinh coast (Pruszak et al. 2001)

iii) ***"Coastal Erosion on a Densely Populated Delta coast – a Case study in Namdinh province, Red River Delta, Vietnam" –*** *Bas Wijdeven, TU Delft, 2002.*

The main objectives of this study were to create an overview of historical and future coastal development of the coastline of Namdinh and to simulate the coastline behaviour over the past 100 years and its interaction with the coastal defences. Thereafter, possible future problems of dike erosion were predicted and mitigating measures were proposed.

The study was carried out based on data and information of some previous studies and reports. A 1D modelling package (WATRON) and UNIBEST packages for longshore sediment transport and coastline dynamics modelling (developed by WL | Delft Hydraulics) were used. Some main conclusion of the study follows:

- The main causes for erosion were found to be in the changing geometry of the Red River system and morphological mechanisms that characterise river mouth development: the abandoning of So river, and the development of the Red river mouth to its present shape.

- The cyclic mechanisms of the Red River mouth cause cyclic alteration of supply and non-supply of sediment of river mouth to downdrift located beaches.
- Construction of Hoa Binh dam on Da river, which is responsible for 53% of the total discharge of Red River system, is the most plausible cause for decrease in sediment supply to the coast.

In the study also discussed in depth about delta forming, side effect of dam construction and deforestation at the upstream of Red River to the erosion problem at Namdinh coast.

iv) "Coastal Morphology, A case study in Province of Namdinh, Red river Delta, Viet Nam" *– Luong Giang Vu, IHE-Delft, 2003*

This is the most recent study which was done as a MSc. Thesis at IHE. The study aimed to state the erosion problem and simulation the coastline changes. By using of the SWAN and Unibest models to simulate the coast of Namdinh and mainly focused on longshore sediment transport problems. The results of the studied can be summarized as following :

- The transition of an accumulating coast into an erosive coast implied a retreat strategy that caused an increase in population pressure in the coastal communes and a decrease of the total land for agriculture, aquaculture and salt mining.
- Besides the reason of the changing of geometry of the Red River system and the morphological mechanisms that characterise river mouth development, the characteristic of nearshore wave climate at the Namdinh coast also is an important factor causing the erosion problem for Namdinh coast.
- The yearly nearshore wave climate is dominated by waves that come from northeast and east directions. These waves have more than 50% of the occurrence frequency of yearly wave climate. Moreover, these waves occur in winter due to strong wind, and approach the HaiHau coast with approximately angle of 45^0 to the shoreline. This implies that these waves contribute a significant part in longshore sediment transport.
- The wave heights of waves that come from the northeast and east increase along the coast of HaiHau district create the gradient in longshore current washing the sediment southward.

The study also found down that the development of Ba Lat estuary does not have significant influence on nearshore wave climate at HaiHau district as mentioned in previous studies.

- The simulation results of 2D wave model SWAN show that the development of BaLat estuary just has some minor influences on wave heights of the wave that come from northeast and east.
- For the waves that come from southeast, south and southwest the development of spits at Ba Lat estuary has shown no influence on wave height.

With SWAN - a 2D wave model - the yearly nearshore wave climate has been extracted at water depth of 10m, and used the longshore computation with UNIBEST and the results has shown a good agreement with observations of net longshore transport direction. This is quite an improvement compared to the wave climates of previous studies, which were derived from 1D wave model.

The longshore sediment transport computations in this study also get improved with comparison to previous study and observations. The net longshore sediment transport at the HaiHau district is approximate 900,000 m^3/year while Pruszak et al. (2001) said 800,000 m^3/year and Bas Wijdeven (2002) posed 600,000 m^3/year.

3.2 Design consideration of sea dikes

3.2.1 General

The function of natural dunes and dikes is to protect the hinterland (population and economical values) against erosion or inundation due to storm surges. The main purpose of a dikes is to fix the land and sea boundaries. Dike is not intended to protect either the beach fronting it. Thus, the dikes neither promote accretion nor reduce the regional trend of the coast to erode. However dikes are constructed for protection of the hinterland under extreme conditions. Dikes are, therefore, one of various forms of coastal protection which may be used on their own and/or in combination with other methods (Pilarczyk, Krystian W. 1998).

The misunderstanding on using of dikes and their possible disadvantages may lead to the disturbance of the natural coastal processes and even the acceleration of beach erosion. However, it should be said that in many cases where the hinterland becomes endangered by inundation (low-lying hinterland as in the Netherlands) or by high rate of erosion (possible increase of sea level rise or cross shore erosion due to storm surge) leading to high economical or ecological losses, the dikes, in this case, can even be a 'must' for survival, whether one likes it or not.

In order to achieve with proper coastal strategy, the decision should always be based on the total balance of the possible effects of the countermeasures for the coast considered, including the technical aspect and economical effects or possibilities. It is an 'engineering-art' to minimize the negative effects of the solution chosen (Kraus and Pilkey, 1988).

3.2.2 Design philosophy

For a certain coastal defence system absolute safety against storm surges is almost impossible to realize because it is impacted by many uncertainty unexpected elements. Therefore, it is much better to speak about the probability of failure of a certain defence system. To apply this method, all possible causes of failure have to be analysed and consequences determined. This method is actually under development in the Netherlands for dike and dune design. The "fault tree" is introduced as a good tool for this aim (Figure 3.6). In figure 3.6, all possible modes of failure of elements can eventually lead to the failure of a dike section and subsequently to inundation. All categories of events which may cause the inundation of hinterland are equally important for the overall safety. However the responsibility of the engineer is mainly limited to technical and structural aspects. In figure 3.6, the correlation between political aim and research and project products is also considered (Pilarczyk, Krystian W. 1998).

In the case of the sea dike, the following main events can be distinguished (Figure 3.5):

- Overflow or overtopping of the dike
- Erosion of the outer slope
- Foreshore erosion leads to consecutive damage of the dikes
- Loss of stability of the revetment
- Instability of the inner slope leading to progressive failure
- Instability of the foundation and internal erosion (i.e. piping)
- Instability of the whole dike

For all these modes of failure, the situation where the forces acting are just balanced by the strength of the construction is considered (the ultimate limit state). In the adapted concept of the ultimate limit state, the probability density function of the "potential threat" (loads) and the "resistance" (dike strength) are combined. The category "potential threat" contains basic variables that can be defined as threatening boundary conditions for the construction. The resistance of the construction is derived from the basic variables by means of theoretical or physical models.

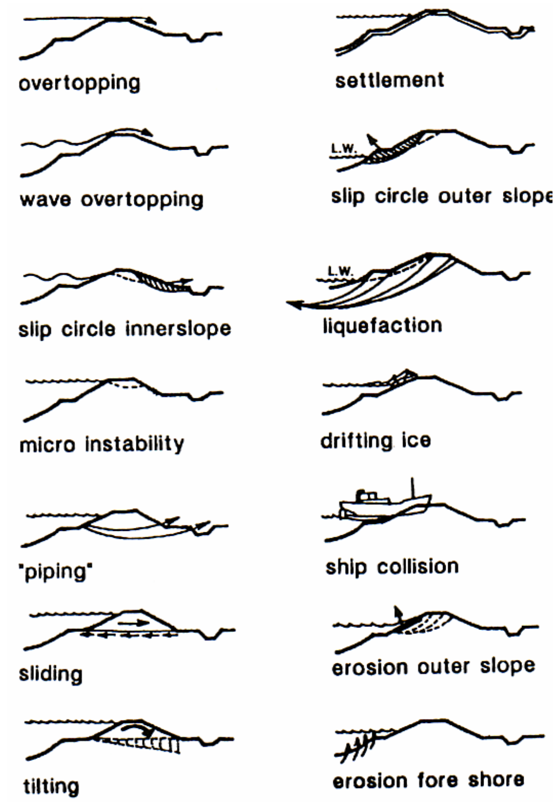


Figure 3.5: Possible failure mechanisms for sea dikes (CUR/TAW 1995)

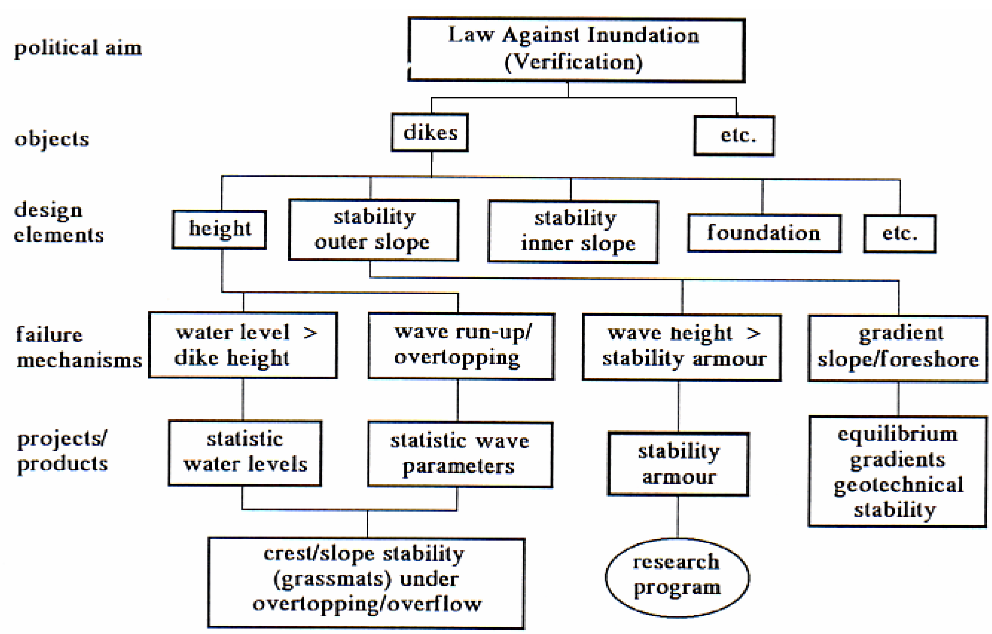


Figure 3.6: Simplified event tree for a dike (Pilarczyk, Krystian W., 1998)

The relations that are used to derive the potential threat of boundary conditions are called transfer functions (e.g. to transform waves or tides into forces on grains or other structural elements). The probability of occurrence of this situation (balance) for each technical failure mechanism can be found by applying mathematical and statistical techniques.

The safety margin between "potential threat" and "resistance" must guarantee a sufficiently low probability of failure. The different philosophies of design approach are currently available in actual design and construction practice. In the Netherlands the following philosophies are available (CUR, 1989, TAW, 1990, CUR/CIRIA, 1991, CUR169, 2000): Deterministic approach; Quasi-probabilistic approach; and Probabilistic approach.

For a fully probabilistic approach, more knowledge must still be acquired concerning all the problems associated with the use of theoretical models relating loads and strength. Studies on all these topics are still going on in the Netherlands. The present Dutch guidelines for dike and dune design follow a philosophy that lies between the deterministic and the quasi-probabilistic approach. While the current latest guidelines for design of the dikes in Vietnam still use the philosophy of fully deterministic.

The ultimate potential threat for the Dutch dikes is derived from extreme storm surge levels with a very low probability of exceedance (approximately 1% per century for sea dikes and dunes, and 10% for river dikes) and equated with the average resistance of the dike without any apparent safety margin. Under these ultimate load conditions, the probability of the dike (sea wall) failing should not exceed 10%.

In the current safety concept, each individual dike section has to resist a certain design water level. If the design water level is exceeded, the flood defence does not fail or collapse directly. Additional requirements give the dike section a reserve or safety margin. The size of that safety margin is unknown.

The evaluation of actual researches in The Netherlands showed that uncertainty of geotechnical parameters led to conservative calculations both during the design and during the flood. To gain more insight into this margin, an improvement of soil investigation techniques and geotechnical analysis is required.

Furthermore, the mathematical description of the various failure mechanisms of dikes is far from perfect. More research in this field, both numerical and physical, is required now and in the near future. Because of the large uncertainties, especially in relation to the structural behaviour of dikes, the monitoring of performance of the dike during a flood may reduce uncertainties.

3.2.3 Design methodology

A general overview of a design process for coastal defence was shown in figure 3.1. For the sea dike the main functions are prevention of inundation and hinterland protection. Based on the main functional objectives of dikes, a set of technical requirements has to be assessed. When designing a dike, the following requirements to be met can be formulated:

1. The structure should offer the required extent of protection against flooding at an acceptable risk.

2. Events at the sea dike should be interpreted from a regional perspective of the coast.

3. It must be possible to manage and maintain the structure.

4. When possible, requirements resulting from landscape, recreational and ecological viewpoints should also be met.

5. The construction cost should be minimized to an acceptable, responsible level,

6. Legal restrictions.

Expansion of these above requirements depends on specific local circumstances. The high dikes are needed for protection of lowlands against inundation while lower ones are often acceptable sufficient in other cases. The material to be use for dike's body should be local material or sufficient low cost if borrow from others. Selection of type and dimension of slope protection must be based on the availability of construction equipments, manpower, and possibility of repairing and maintenance. The cost of construction and maintenance is generally a controlling factor in determining the type of structure to be used. The starting points for the design should be carefully examined in cooperation with the client or future manager of the project.

3.2.4 Boundary Conditions and Interactions

3.2.4.1 Boundary Conditions

a. Hydraulic boundary conditions

Under consideration of function of coastal defences, the loads will be mostly due to the actions of water waves in term of long and/or short waves.

The determination of hydraulic boundary conditions is followed by the flow diagram given in figure 3.7. For a coastal defence structures like sea dikes, water levels are governed by tides and winds. The most complex situation occurs at coastal shores, where water level fluctuations can assume many forms.

Under the engineer's point of view for design of sea dikes, the design water levels and design wave characteristics (wave height, period, and approached angle) are primary element to establish the hydraulic boundary condition. The magnitude of water level and wave height in order of decimetres are acceptable.

The design high water level should be considered as the maximum water level during a tidal circle (19 years), which is contributed by tidal level, wind setup, surge level and other types of water level fluctuation. At the location where the observation of water level sufficient long the design water level can be determined from exceedance curve with a certain design frequency.

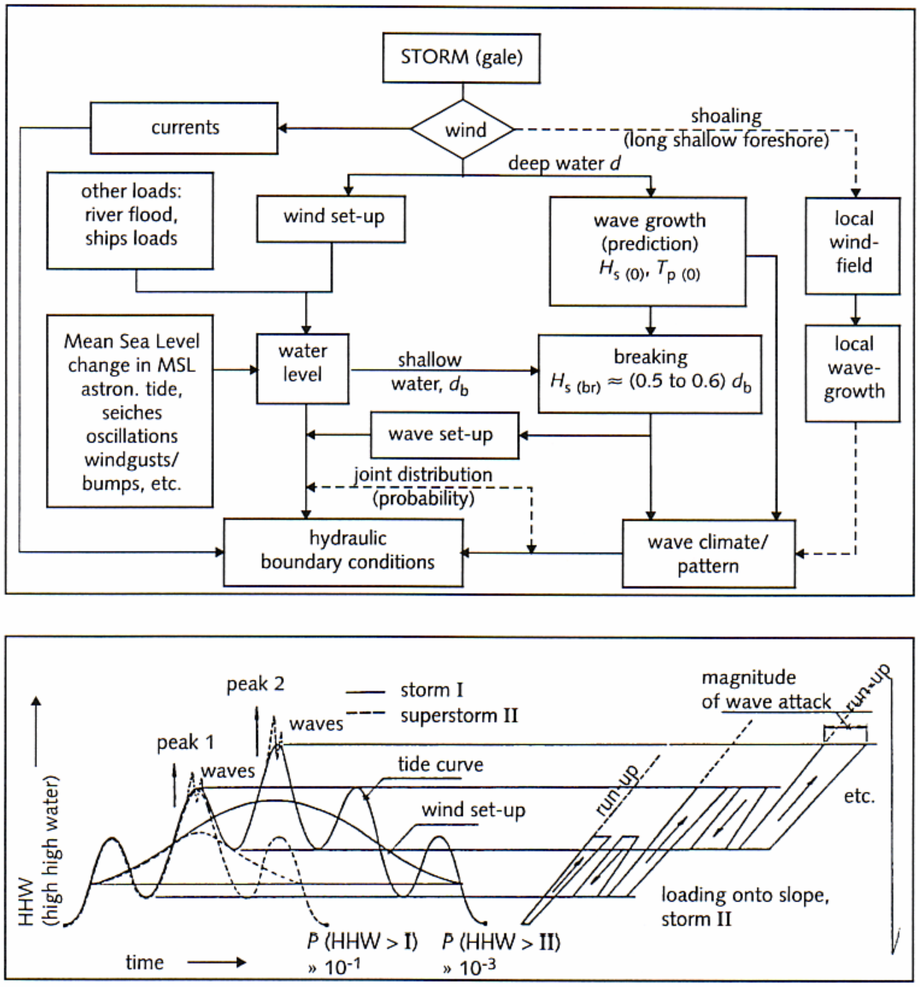


Figure 3.7: Overview of determination of hydraulic boundary conditions

(Pilarczyk, Krystian W. 1998)

Design wave height and wave periods are the most important parameter of design wave characteristics. The design wave height for sea dikes is the local wave height, which can be derived from one of the following ways:

- Transformation of significant deep water waves to the toe position of the dikes.

Applying this method in the case of near the research area there is offshore wave observed station which provides sufficient long record in order of several years or even decades (long term records).

- Prediction of significant waves based on wind data with hind-cast methods.

There are several methods which give the prediction of wave height and wave period by using wind data, such as method of Breschneider 1952, then developed by Sverdrup and Munk. The background of the hind-cast method is that they considered the wave height and wave period is a function of the wind-velocity, the fetch length and the water depth.

- Using depth-limited wave height at the position of the toe.

The local wave height in front of the dikes can be determined by the relation to the local water depth. It is so called depth-limited wave height. For most of the cases the local significant wave height can be determined by 0.45 to 0.55 time local water depth.

One should be note that for design of sea dikes as well as onshore structures, the depth limited wave height is a good and simple way for estimating and/or verifying the design wave height which derived from other methods.

b. Geotechnical conditions

Geological aspects and geotechnical conditions are important for construction and stability of the structures. This must be known in advance during design process by geotechnical investigated activities.

The first step is to organize and design site investigations. The field programme as part of the site investigation is complemented by laboratory testing and geotechnical calculations. The last and perhaps most difficult step is the integration of the result of the investigations and structural design, resulting in the final foundation design.

The quality of investigation must be accepted by the national related standards (which may derive from international standards). A high-quality investigation must be economically efficient in the sense that the cost of the investigation must be money well spent.

c. Construction materials

A large number of materials may be used in various forms in the construction of sea dikes. These can be: sand, gravel, quarry rock, industrial waste products (slags, minestone, silex from cement industry), clay, timber, concrete, asphalt, geotextile, etc. All these materials have to meet some structural and environmental specifications which are usually regulated by the national standards. Furthermore the selection of materials has to be considered their availability and technical feasibility during construction.

3.2.4.2 Processes and interactions (Pilarczyk, Krystian W. 1998)

a. Loading zones

For coastal areas there is a correlation between the water level (tide plus wind set-up) and the height of the waves, because wind set-up and waves are both caused by wind. Therefore, the joined frequency distribution of water levels and waves seems to be the most appropriate for the design purposes. For sea dikes the following approximate loading zones can be distinguished:

Zone I: the zone permanently submerged (not present in the case of a high level "foreshore");

Zone II: the zone between MLW and MHW; the ever-present wave load of low intensity is of importance for the long-term behaviour of a structure;

Zone III: the zone between MHW and the design level; this zone can be attacked heavily by waves, but the frequency of such an attack reduces as one goes higher up the slope;

Zone IV: the zone above design level, where there should only be wave runup.

In principle, under a certain conditions the persistent character of the wave-attack of the slope protected structures as well as the dike is different by loading zones. This difference leads to that the quality of the seaward slope can, prior to the occurrence of the extreme situation, already be damaged during relatively normal conditions to such a degree that its strength is no longer sufficient to provide protection during an extreme storm. The division of the slope into loading zones, on one hand, has a direct connection with the safety against failure of the revetment and the dike as a whole. On the other hand, this division provides different applications of materials and execution and maintenance methods for each zone.

b. Load-strength concept

Once the hydraulic design conditions have been established, actual design loads have to be formulated. For a given structure many different modes of failure may be distinguished, each with a different critical loading condition. Each of these failure modes may be induced by geotechnical or hydro-dynamical phenomena. This section is restricted to the stability of the front slope which related to hydro-dynamical processes.

Starting with the hydraulic input (waves, water levels) and the description of the structure, external pressures on the seaward slope are determined. Together with the internal characteristics of the structure (porosity of the revetment and secondary layers) these pressures result in an internal flow field with corresponding internal pressures. The resultant load on the revetment has to be compared with the structural strength, which can be mobilized to resist these loads. If this strength is inadequate, the revetment will deform and may ultimately fail.

The phenomena may be relevantly contributed by three components of the system: water, soil and structure. The interaction between these components can be described using three transfer functions. More detail sees Pilarczyk et al., Dikes and revetments, 1998.

1. The Transfer Function I: from the overall hydraulic conditions, e.g. wave height H, mean current velocity U to the hydraulic conditions along the external surface, i.e. the boundary between free water and the protection or soil, e.g. external pressure P.

2. The Transfer Function II: from the hydraulic conditions along the external surface to those along the internal surface, i.e. the boundary between protection and soil. The hydraulic conditions along the internal surface can be described as the internal pressure.

3. The Transfer (Response) Function III: the structural response of the protection to the loads along both surfaces.

Information about these functions can be obtained by means of measurements in site and scale model tests. If quantitative knowledge of the physical phenomena involved is available or if there is enough experience available, then mathematical models or empirical formulae containing information are formulated and referred to as "models". All three Transfer Functions can be described in one model, or individually in three separate models, depending on the type of structure and the loading. The distinction between the three functions here mainly serves as a framework to describe the different phenomena that are important for the modelling.

Due to the fact that, in many cases, the various processes cannot be described yet therefore a "black box" approach is advised to follow. In which the relation between critical strength parameters, structural characteristics and hydraulic parameters are obtained empirically.

3.2.4.3 Consideration of slope protection

The technical feasibility and dimensioning of coastal structures can actually be determined on a more solid basis and supported by better experience than in the past. Often, however, the solution being considered should still be tested in a scale model since no generally accepted design rules exist for all possible solutions and circumstances.

The types of slope protection for the sea dikes or dunes (revetments) which are presently being studied are various by variety of applied materials and types of elements. Each kind of revetment corresponds to some critical modes of failure which driven by correspondent determinant loads and the required strengths (see also Pilarczyk, Krystian W., 1998). Most of the determinant loads are derived by waves while the required strength (resistance) of the protection is derived from friction, cohesion, weight of the units, friction between the units, interlocking and mechanical strength. The classical slope revetments may be divided mainly into four different categories:

- Natural material (sand, clay and grass)
- Protected by loose units (gravel, riprap)
- Protected by interlocking units (concrete blocks and mats)
- Protected by concrete and asphalt slabs.

For difference kind of slope protection, the critical loading conditions are different by difference of strength properties. Maximum velocities will be determined for clay/grass dikes and gravel/ riprap, as they cause displacement of the material, while uplift pressures and impacts, however, are of more importance for paved revetments and slabs, as they tend to lift the protection. As these phenomena vary both in space and time, critical loading conditions vary both with respect to the position along the slope and the time during the passage of a wave. Instability for grass/clay and gravel/riprap will occur around the water level, where velocities are highest during up and downrush. Moreover, wave impacts are more intense in the area just below the still water level. For designing slope protection the engineer must be understood qualitatively and quantitatively of the relation of the critical loading conditions and correspondent strengths for applied type of revetment.

Optimization of slope protection is the same meaning to optimization slope stability and availability of implementation. For a certain hydraulic boundary conditions the optimization must be considered the optimal shape of profile, applied cover elements and their size, and the relations between them.

One can be easily to say that the let’s design revetment by the equilibrium slopes with certain type of material for the given boundary conditions. Theoretically, it is true to say so. The principle is that the wave forces on a plane (continuous) slope are distributed rather unequally (the high wave-impact area near the water level, the intermediate uprush area and the low-attacked area beneath the point of breaking), and the wave action on relatively fine materials indicates that nature tries to distribute the forces equally to provide equilibrium S-slopes. This S-slopes can be applied in the design of the shape of slope protection which leads to the application of smaller protective units and, off course, more stability than in the case of a plane slope. However, firstly it is difficult to know in advance accurate S-Slopes. Precise determination of S-slopes can only be obtained by time-full scale model (scale model of 1:1). Secondly for practical reasons the 'optimal' shape will be schematized to a trapezoidal profile. By selecting a proper position of a berm below the design water level and a proper width of the berm, the wave forces will be distributed in such a uniform way that the same material can be used along the whole profile. In this case, an increase of stability (50% or more) can be realized by a berm with a width equal to 0. 15 times the wave length and situated (0.5 - 1.0) times the wave height below the design water level. Based on the results of various studies, the indicative design guidelines have been prepared for riprap bermed-slopes and toe protection (Pilarczyk, 1990 and CUR/CIRIA, 1991) and for rock or block revetments (Van der Meer et al, 1995).

Furthermore for optimization of a plane slope protection the consideration should be paid to the slope angle, size and weight of protective elements, local capacity and availability of constructional equipments and man power during construction and maintenance processes. For instance, the outer slope of a sea dike section is protected by a two layers riprap revetment. The design wave height is 1.50 meters. Roughly, the required dominant size of rock D_{50} is 1.50 meters for slope steepness of 1:2. The range of rock size should lie between 1.10 to 1.80 meters (weight of such rock about 3.5 tons to 16.0 tons). Then total required thickness is 3.60 meters. For construction process the placements of rock can not be done by man, instead of that, it is required the construction equipment with the capacity of about 20 tons. It seems to be no problems for the initial construction because the contractor must have enough, and/or for the rich regions which easily to have such construction equipments. Unfortunately, in developing country the sea dikes usually very far away from the cities and there are not too many such construction equipments which are always busy with the new construction works. In this case it is difficult to maintain if there is any damage after a storm. Instead of applying 1:2 for slope steepness, the slope of 1:4 is introduced which requires the size of rock from 0.40 to 0.60 meters (0.2 to 0.57 tons) which can be handled by men. Therefore after a storm the maintenance can be done by a group of men in order to recover the rocks in to it right places

CHAPTER 4 POSSIBLE FAILURE MECHANISMS OF NAMDINH SEA DIKES

4.1 From historical development of the system to Future prediction.

4.1.1 General.

Namdinh province has a long coastline. The sea dikes have been the prevalent defensive system protecting the coastal areas from sea water flood and waves attacks. This system was established and has been developed continuously for a century ago. It suffered from many times of heavy damage, failure and then breaches by attacks of severe storms which often accompanied surges and high tidal level.

Several reports are available which describe the development of sea dike system at Namdinh coast. These reports also include some major events of dike's breaches and reconstruction of the dikes after breaches since 1890. However, the reports referred to the most impressive events from 1890 to 1975. It should be indicated that during this period there were no observation and measured records. Therefore the understanding of long historical development of the dikes can only be carried out from description under text form documents. Since 1972 there has been observation therefore the situation can be analysed in more detail. From 1972 to 1990 every 4 to 5 years there was an observation. Since 1990 the measurements of cross shore profiles were performed every year.

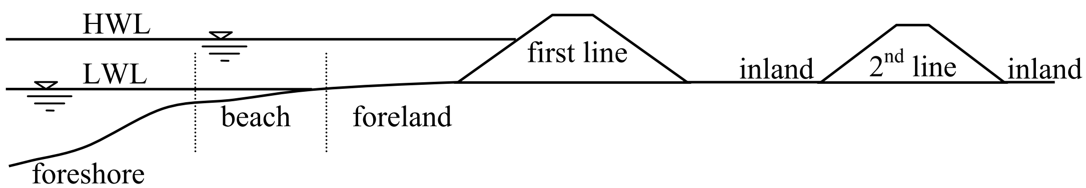


Figure 4.1 Shoreline definitions.

The historical records show that there were various kinds of failure modes presented at Namdinh sea dikes. The reason for that could not be one on the other hand the failures were caused by many reasons. The followings could be considered as the dominant causes:

- Heavy erosion of near shore with continuous process led to lowering the sand beach and foreland in front of the dikes. As the consequences the toe of the dikes could not be stable after some time. The failures of the toe then resulted in series of consecutive damages of upper components of the dikes.
- Increase in water depth in front of the dikes due to foreshore erosion caused higher waves height (compared to design wave height) attacking directly to the dikes. Therefore the dike's components could not withstand longer.
- Changes in boundary conditions during operational time. Mostly the changes are hydraulic boundaries such as onshore and offshore wave parameters, long shore currents and sedimentation processes, water level due to sea level rise.

- Lack of knowledge and theory in design and construction processes. The previous design, which was based on experience only, may not be sufficient to the boundary conditions.
- Economic limitation led to poor in operation and maintenance

4.1.2 From historical analyze of dike's development to future prediction

The historical failure of the dike system in Namdinh province from 1890 to 2000, according to the records, can be summarised as the follows:

4.1.2.1 Period from 1890 to 1971:

- In 1890, after the severe storms all dike sections were destroyed along HaiHau coastline. Large area was inundated and flooded with considerable damages. Later new dikes system was constructed and placed 500m inland from previous coastline.
- In 1902, the dikes system which was constructed after 1890 breached completely. This resulted in serious losses. About ten thousand people were killed and local people were endangered by a serious famine.
- A new defensive system was built after a number of years. Aiming at getting a safer and more reliable dike system, 2 defensive lines were constructed. The first line was primarily dikes which faces direct to the sea. It provided against storms and flood. The second one then planned to provide for all possible contingencies which could occurred in the case of the first line came to breach. The whole system was shifted about 350m landward.
- In 1925, the first dikes breached over the length of 3000m approximately along Vanly section. Therefore at this section the second line became the new first line and

- In 1944, the dike sections from Haithinh to Haichinh of both first and second line were breached (of about 12 km length). Due to that the inundation area was about 3200ha. The whole dike system then shifted about 350 m inland.

- From 1945 to 1971, no dike breach occurred. Exceptionally, the outer slope of some sections was damaged. However all the damages could be repaired in time and the local people did the operational maintenance seasonally. Therefore the dike system could exist longer.

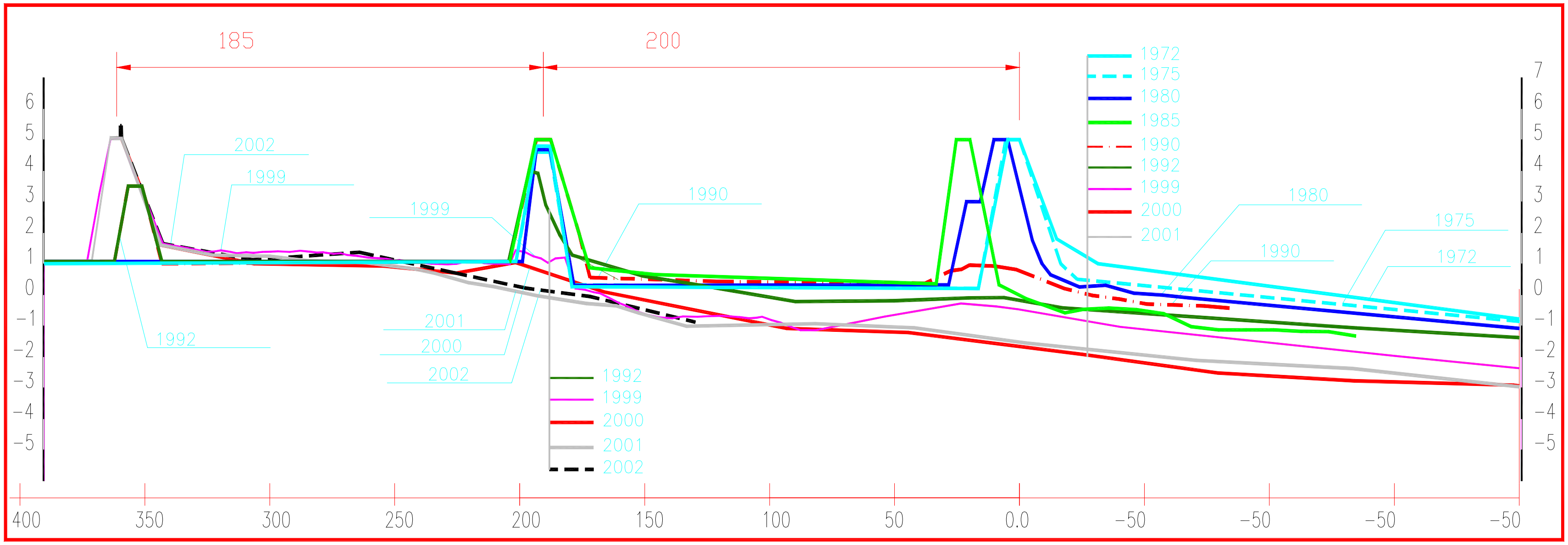


Figure 4.2a: Overlapped cross-shore profiles at HaiTrieu section over last 30 years

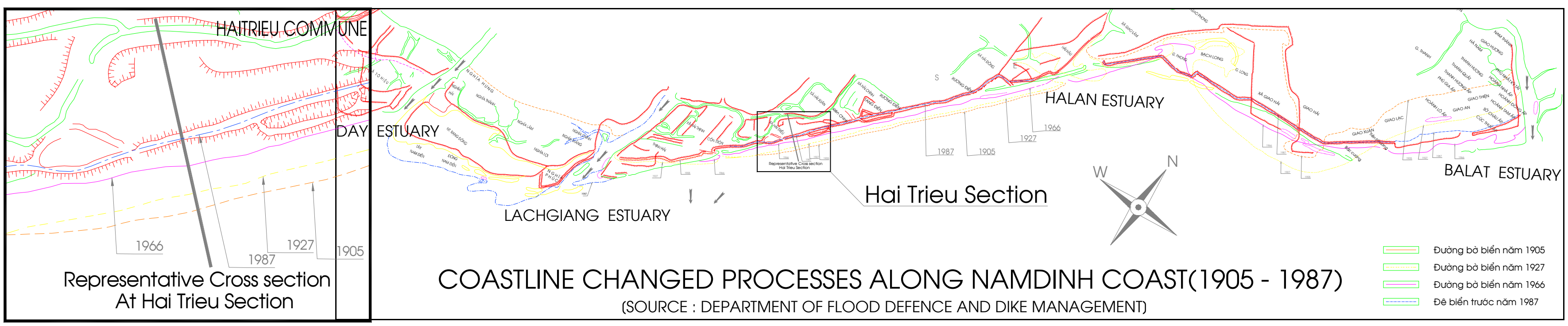


Figure 4.2b: Position of HaiTrieu section and representative cross section.

4.1.2.2 Period from 1971 to 2002:

During this period the were 6 times of dike breached along HaiHau coastline over the total length of about 8300 m. The maximum retreat of coastline (indicated by the position of first defensive line) during 30 years is about 400m.

In order to analyze the historical development of the dikes along this period, a section of sea dikes at HaiTrieu committee is introduced as a representative section for HaiHau sea dikes. This section is considered like the most serious place which suffered from severe erosion of beach and foreland and the dikes breached frequently. Moreover at this section the observation data has been collected since 1972. The cross-shore profile which was measured one after one to 5 years shows not only cross-shore development but also the positions of the defensive lines from 1972. Therefore analysis of historical development based on the observation data will be more reliable.

In 1972 the dike system at HaiTrieu had 2 lines of defence. The first line was in good condition with crest level of +5.0 CD, and outer slope of 1 over 3 while inner slope of 1 over 2. Composition of dike’s cross section was sandy clay body and moderated clay for outer armour layer. The toe of the dike was protected by extending under part of the outer slope with a flatter slope of 1 over 10. Foreland and beach in front of the dike was at level from -0.5 to +1.0 and around 150 m wide. The second line stood at 200m landward of the first line. The dimension of cross section was smaller then that of the first line. The crest level was at +4.5 m CD, outer slope is one over 2 and inner slope is one over 1.5. The profile of cross section is shown on Figure 4.2, *line label 1972*.

Three years later, in 1975, the erosion of near shore zone started. The foreland and beach along the dikes was lowered approximately 40 cm annually. The erosion depth was even more near the toe of the dikes, about 65 cm. Due to this erosion the toe was in danger of instability. The erosion of the lower part of outer slope was started. The first line of defensive system was in threatened. The profile (line label 1975, Figure 4.2) was deeper offset in comparison to 1972’s.

During period of next 5 years, from 1975 to 1980, the erosion became more heavily. Continuous erosion occurred near water line due to the actions of waves leading to the loss of body’s material. In front of the toe the sand was taken away forming scour holes. The cross section of the dikes was getting smaller and thinner. Instantly the first line was upgraded by the local people. However people reinforced the dikes by widening the cross section at inner side instead of reinforcement the outer slope or even they did not care about the occurring of foreshore erosion. The cross section then looked like *line label 1980* in Figure 4.2

In 1984 the erosion of near shore zone was still going on and the scour holes were getting deeper. The toe of the dikes was collapsed first after that sliding mode occurred as the failure of the outer slope. At the end of storm season in 1984 the first line was breached over the length of 1500 meters.

During quiet period of the sea in 1985 the breached gap was closed by new first dike section temporarily. At the same time the second line was upgraded for intention of becoming the first line later. The temporary first dike (closured section) was constructed at position of the old dikes basically in order to reduce the risk at the second line and kept dry area in front of that for being working site. After upgrading, the second line now could become the first one while the old first one was considered temporarily and waiting for the occurrence of breach. Establishment of the new second line was started.

From 1985 to 1990 the first dike section, which was constructed temporary in 1985, had been getting weaker. At the end of storm season in 1990 the dike cross section remained only a small area with the shape of triangular (*see line label 1990*). The material was spread out by interaction of waves and currents. The beach reached the toe of new first line and beach averaged level was about +0.50. The averaged lowering rate of beach was about 0.25 m per year. End of November 1992 due to a severe storm the beach erosion increased and scour holes appeared in front of the toe. The dike's body was starting damage. At some places the sliding failure of the outer slope occurred. People tried to maintain the outer parts of the dikes by filling some rock at the toe and reinforcement outer slope. The dikes then could withstand until November 1995. After a storm with wind strength of 10 Beaufort the dike section mainly collapsed. The remained part had a triangle shape and it was not considered as a dike any more. At that moment the position of first dikes was shifted to the second one's which stood 200 meters behind.

During the period from 1996 to 2000, people again were busy with upgrading and construction of the dike system. In 1999 the rest part of the old first dikes was gone out completely. Severe erosion was still occurred therefore the foreland was narrower. The sea had reached at the position of old first line. The level of the foreland zone was round -1.0 m.

The upgrading works were finished in 2000. New first line of dikes had crest level of +5.5m. The outer slope was one over four which was protected by rock revetments. The toe of the dikes was at level of +0.5 m with one or two lines of cylindrical concrete block. In front of the dikes the protection of scour holes was provided, see also report of FAO/UNDP 525 project, design documents for upgrading Namdinh sea dikes.

After 3 storm seasons from 2001 to 2003 the defensive system worked well in good condition. However the observation showed that the beach was still getting narrower and deeper year after year.

The retreat of coastline during the period of 30 years is shown in Figures 4.3. In this Figure, the retreats of foreland levels of -0.50, 0.0, +0.50 meters (+MSL) are indicated. The based point is the position of foreland level 0.0 in the year 1972.

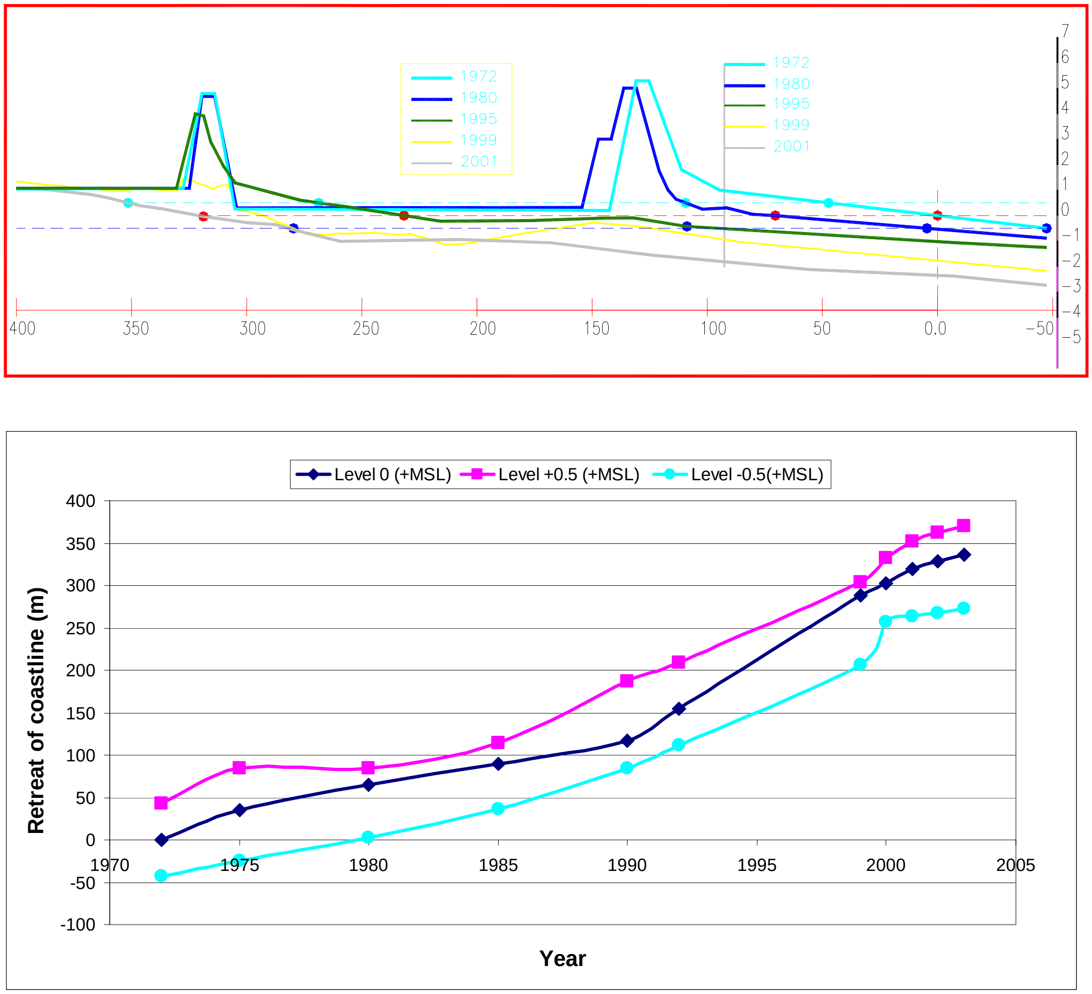


Figure 4.3: Retreat of coastline during from 1972 to 2002

4.1.2.3 Summary

The planned development of Namdinh sea dikes started a long time ago. During period of 65 years from 1890 to 1971, the dikes system had to shift into inland about 850 meters by 6 times of dike moving. Especially for the period of 30 years from 1972 to 2002, there was 6 times of dikes breach and 3 times of reconstruction of the dikes which shifting inland. The retreat of coastline during this period was around 400m inland. The dike breach and retreat of coastline for the time being because of the mentioned reasons. These processes can be summarized as following:

Firstly due to the heavy erosion, the foreshore in front of the dikes was going deeper year by year and the foreshore slope was becoming steeper. As the consequence, the sand near the toe of dikes was taken away and forming as scour holes. It caused instabilities of the toe first as well as instability of the revetments and outer slope of the dikes. Then after sometime when the flat reached at certain depth, the dikes were damaged and destroyed.

Secondly the outer slope of the dikes were not provided proper protections while the water level during storm is often accompanied with a surge and reached at the level of +2.0 (crest level of the dikes was at +5.5). Under these conditions the waves always attacked and broke directly on the non-protected outer slope of the dikes. Therefore the erosion of outer slope happened easily and the dike's body was getting smaller after each storm.

In addition the dikes could not be stable due to attacking of the higher wave height comparing to the design wave height. It could be explained that when the heavy erosion occurred, the water depth was getting deeper. Figure 4.2 shows that after 30 years, the bed level at the same position was 2m deeper. This, as the consequence, leads to the larger water depth and the higher wave height which could occur near shore in front of the dikes. Therefore the dikes body and their structures were in more dangerous situation.

In the research area, the dikes system was usually designed and constructed by two lines see Figure 4.1. The first line was considered as the main protection line. The second line which located about 200m land ward side of the first line was provided against storm and flood if the first one breached. However, the first line of protection could normally stand more or less 5 to 10 years only then collapsed. After that the second line became the first one and people started constructing the new second line again.

Future prediction

Based on the above analysis, if no proper measures for upgrade and protection of the dikes are undertaken, the situation will go on by similar trend of the last period. After every 10 years the retreat of coastline will be around 150 meters inland, see also Figure 4.3. According to the current trend, the location of the dikes will shift 200 meters landward after every 10 years.

4.2 Possible failure modes of NamDinh sea dikes.

In this section various kind of failure modes, which did occur and may occur, are mentioned. For each of those, the failure mechanism will be investigated and analyzed. Based on that the possible consequences will be indicated theoretically when the failure mechanisms occur.

4.2.1 Hydraulic related failure modes

4.2.1.1 Wave run-up and wave overtopping.

The failure of the dikes due to wave run-up and overtopping, may occur when there are presenting the following possible modes.

- The crest height of the dikes is insufficient at some places (sections) the water level even exceeds the actual crest.
- Wave run-up at the high tides compiles with surges resulting in high water level which can pass the crest of the dikes.
- Overtopped sea water is too much, exceeding the critical value of wave overtopping.

Due to wave run-up and wave overtopping at on an insufficient dike crest level, the following failures could be occurred:

- Damages of dike crest and crown wall by wave run-up at DWL accompanies to heavy storm.

- Erosion of crest sides.

- Erosion of inner slope due to too much wave overtopping.

- Damage of outer slope protection due wave impact or erosion at the toe.

- Inundation at lee side due to too much of overtopped sea water.

- Damage of upper part of revetment due to the return flow of overtopped sea water.

- Washing material of filter layer, where the dike body was not well protected by cover layers.

The Figure 4.3 shows the impression of failures at a dike due to wave run-up and wave overtopping. This is an evidence of dike damages at Namdinh, which took during site visits, recently. The damages of the dike in that picture included failures of crest wall and upper parts of revetments at Haithinh section in HaiHau, Namdinh.

It is clear that, the most important parameters to avoid these failures due to wave run-up and wave overtopping are crest level of the dikes and the strength of crest wall. However, the questions are: how was the crest level determined for existing dikes in Vietnam; what was the contribution to that level; how is the quality of the crest wall, does it meet the design requirement. Finally based on the analysis the answer will be given for the question of what should be improved for more accuracy design and proper construction of the dikes.

Figure 4.3: Damages of crest-wall at Namdinh seadike at Haithinh section

The first component which mainly contributes to crest height of a dike is design water level. It should be the highest water level which may occur at the location. Normally the design water level includes mean water level, tidal level, storm surges, wind setup, wave setup ... The design water level must be an extreme water level corresponding to design frequency and design life time of structures.

The second important component is wave run-up level. This depends on wave's characteristic in front of structures, type of outer slope of structures, and also foreshore feature (shallow foreshore or deep foreshore). The dominant parameter which gives the most contribution to wave run-up is wave height at the location near the toe of the dikes. It is so called local design wave height for the dikes. In order to get a proper wave run-up level, one should pay much attention to how the design wave height is chosen.

According to Dutch design codes, the crest height does not just depend on wave run-up or overtopping. Account must also be taken of a reference level, local sudden gust and oscillations, setting and an increase of the water level due to sea level rise. However, the contributions which can be influenced by dike structures itself are wave run-up and overtopping (CUR 169, 2001).

Wave run-up and overtopping on a dike slope depend on the outer slope, which can consist of variety of materials by different roughness. More over the slopes are not always straight, if there is a berm, the upper and lower slope may have different angles. The berm width is also a parameter which can influence the magnitude of the run-up and overtopping of incident waves, and the impact on slope protection.

All sections of the sea dikes in Vietnam which were constructed before 1995 were designed by applying of old design codes or some even based on only experiences of designer. In the design practice at that time various formulae were used for prediction of wave run-up. However there was no recommendation which formula was the most actual and could be used. The formulae which were used the most frequently, were two Russian formulae [TOZV-4-96050].

The comparisons by applying Vietnamese and Dutch codes for determined crest height as well as wave run-up and overtopping will be treated in chapter 5.

4.2.1.2 Failures of inner slope

The failure of inner slope could happen when the dikes were under heavy wave attacks (in severe storms) resulting in higher wave run-up and larger wave overtopping comparing to design values. When the overtopping discharge is high enough, it can wash the material out of the grass mats. If the overtopping remains long enough there will be losses of considerable amount of dike's body material.

The mechanism is that the overtopped sea water passes dike crest exceeding the design value. The water will form an overflow flowing on the inner side of the dikes with a certain velocity. Due to the interaction of the overflow, surface soil (material) layer of dike body will be taken away by the flow. Also due to overflow, the inner slope will be saturated which may leads to local sliding. Normally after a severe storm the inner slope will be eroded therefore the dike body will be weakened. If there will not be any in-time proper maintenance, the consecutive damages of the dikes will be followed, such as local instability of slope, macro instability of dike body…

For the existing sea dikes in Namdinh, there were not any criteria of overtopping during design processes. When severe storm, which more or less likes design storm, occurs it often causes a lot overtopped water via the dike crest. On the other hand the inner side of the dikes is not well protected. Therefore the damages of inner side usually happen.

As understanding from the current situation, the damages of inner side of Namdinh dikes were causes by insufficient of design crest level, which based on run-up criteria only. Further analyses about determination of design crest level will be presented in chapter 5. The consideration of overtopping criteria will be taken.

In addition, some other kind of external loading on the dikes should be considered during design processes. The heavy rainfall may be the cause of erosion and instability of inner slope. While the transportation and other men activities should be accounted for when looking at the instability of the dikes.

4.2.1.3 Failures of outer slope

The outer slope of the sea dikes is face directly to the sea. Under the action of heavy waves the dike slope is usually eroded. This is so called storm erosion. For the dikes without slope protection, the erosion is quite similar to erosion of dunes. The erosion of outer slope often starts around position of still water level. After more actions of severe waves the erosion area increases. The erosion process is going on until the shape of outer reaches an equilibrium shape. However for the case of man made dikes, the area of cross section never big enough for being the equilibrium profile. Normally when the heavy erosion occurs during period of high water level, the other failures will happen after that. Then the dike can no more stand.

In Namdinh, the dikes were suffered a lot from erosion of outer slope whenever the dikes have been protected by the revetments or not. Normally, the existing revetments did not cover all the length/area of outer slope to the dikes crest. Therefore the erosion takes place above the revetments as staring point, then this process continues and more material of dikes body is taken out. This leads to threatening the safety of the dikes as well as the revetments.

Figures 4.5 shows the heavy damage of the dikes at Haichinh section. According to the Department of Dike Management, first the erosion occurred at the top of riprap revetments. After a design storm, the situation was getting more seriously. Then the dike body and revetment were collapsed after some days. Referring to the situation it is possible to say that the final failure occurred under sliding mode of outer slope.

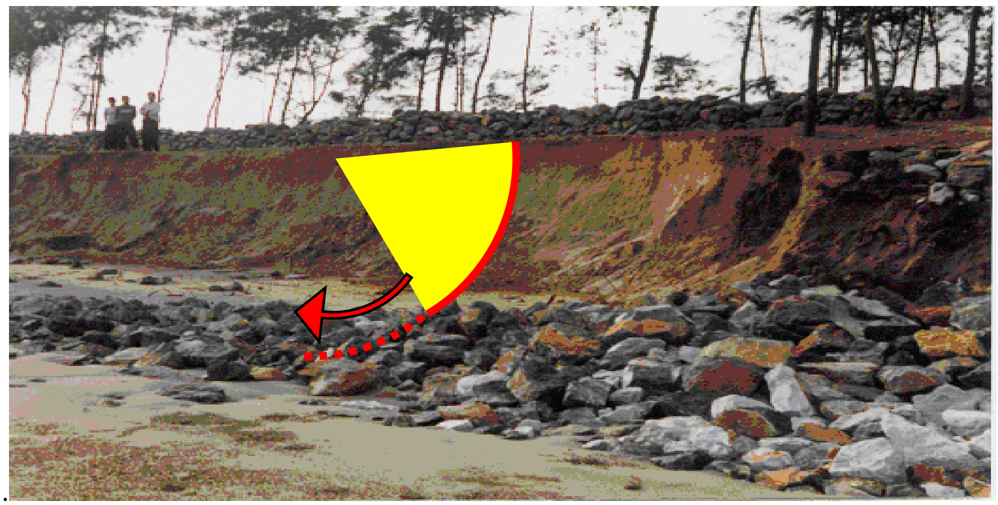


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Figure 4.5: Heavy damage of revetment and outer.

4.2.1.4 Foreshore erosion

Foreshore erosion in front of the dikes affects contributes to the working state of the dikes. For many parts of coastline in Vietnam, especially for Namdinh coastline where the severe erosion takes places, the dikes were suffered from heavy damages and even collapsed after some years of complete construction. Due to the foreshore erosion, the sand near the toe was taken away. After that the toe and the outer part of the dikes could be collapsed. Further more the water depth at the toe of the dikes was bigger, resulting in higher wave height attacked to outer slopes. As the consequences, there was a series of damages of the dike's components like cover layer of revetments, erosion of outer slope then destruction of dike's body.

For the present design of sea dikes in Vietnam, this kind of failure mode has not appeared yet. It is necessary to say that the foreshore erosion should also be taken in to consideration for design of sea dike by providing a proper type of protections in front of the toe of the dikes. In other word, the foreshore should be stabilized in front of the dikes. For the existing dikes the observations of scour hole should be made then suitable measures should be given in time. The dimensional parameters for the protection can be carried out from both physical models and analytical models. In chapter 5, the preliminary determination of these parameters will be indicated by applying the analytical method.

Foreshore erosion causes direct and indirect effects to the safety of the dikes and revetments. It can be explained that, firstly when the foreshore erosion takes place, the bathymetry of the sea bed in front of the dikes changes as well. This leads to the changes of hydraulic boundary condition of design situation comparing to current situation. Consequently, the loads on the dikes change (usually increases). Secondly foreshore erosion occurs near the toe of the dikes. It concentrates and forms scour holes there. When the scour depth reached at certain depth (near the level of toe foundation) the toe structure can not be stable under action of waves and currents. The failures happen to the upper structures when the toe is collapsed as consecutive consequences.

At the location of severe foreshore erosion in Namdinh, the dikes system had to shift inland a hundred meters after a number of years. See section 4.1 also.

4.2.2 Geo-technical related failure of dike's body

4.2.2.1 Instability of inner and outer slopes

For soil sloping structures, the most important failure mechanisms are circular shear failures of outer and inner slopes (slip circle failures). It happens if the actual shear stress along a potential failure planes exceed the shearing resistance along that plane. In most cases the circular planes are considered, see Figure 4.5. The exceptional cases are for cross section where planes with a deviating shape are more likely compiled planes.

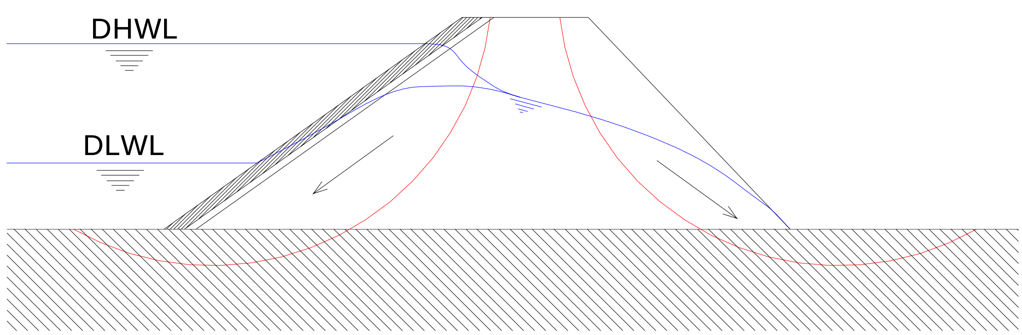


Figure 4.5. Erosion of outer slope leaded to failure of dike body and collapsed revetment

The slopes of the dikes may not stable due to the following reasons:

- Weak resistance at the lower part of the dikes due to the foreshore erosion
- Reduction of the strength of materials during the life time
- Reduction of the strength of saturated materials during heavy rainfall, storm surge and high tides periods.
- Influence of rapid draw-down of sea water level.
- The core materials may not meet the design requirement.
- Constructed quality of dikes may not under proper management
- Heavy transportations on the dikes.

For a representative cross sections, the assessment of slope stability will be investigated in chapter 5 by apply Bishop Method and finite element method.

4.2.2.2 Local instability

These failures may occur at somewhere the working state of the dike's material exceeds its ultimate critical limit state. The local instability of an inside parts of dike's body is probably caused by exceeding of stress-strain states. It is considered reaching the critical state when the Morh circle reaches Coulumb criteria and the stresses lie on the surface of the Coulomb failure envelope. This is so-called plastic point. Moreover in some practical problems, an area with tensile stresses may develop. This behaviour is so-called tension cut-off. In addition the instability of outer grain on surface of the slope due to high concentration of groundwater flow and high hydraulic gradient is considered.

These failure modes will be investigated for Namdinh sea dikes by analysis of the stress- strain state and calculation of seepage (groundwater flow) of the representative cross section. Then we can state whether or not the failures will present.

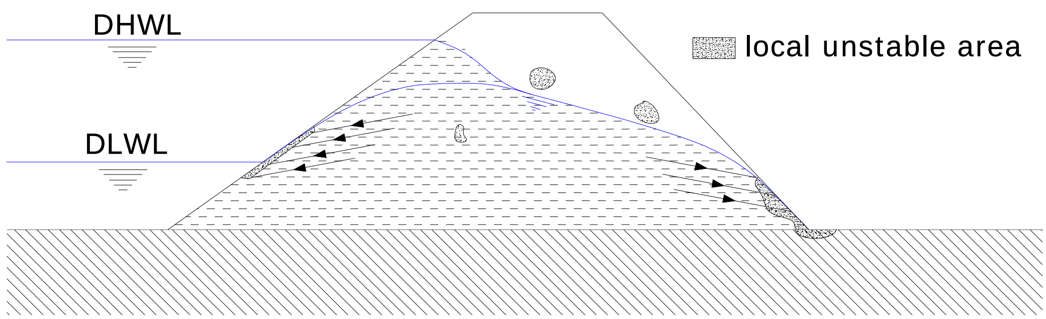


Figure 4.6: Possible local instability due to exceeding critical limit state.

4.2.2.3 Piping

Piping can be described as a concentrated outflow of groundwater on the landside at high seaside water levels, see Figure 4.7, where the velocity of the out-flowing water is such that soil particles are carried along and cavities and tunnels originate due to receding erosion which threatens stability[TAW-1999].

In addition a form of concentrated seepage through a section of soil layer which is either more permeable than its surrounding or is subjected to a particular high hydraulic gradient. Concentration of flow may lead to transport of soil particles and loss by regressive erosion. Once transport is established the hydraulic gradient will increase and transport more particles resulting in an erosion pile.

The probable failure due to piping will be checked by the recent methods then we can state the safety of the dikes concerning with this aspect.

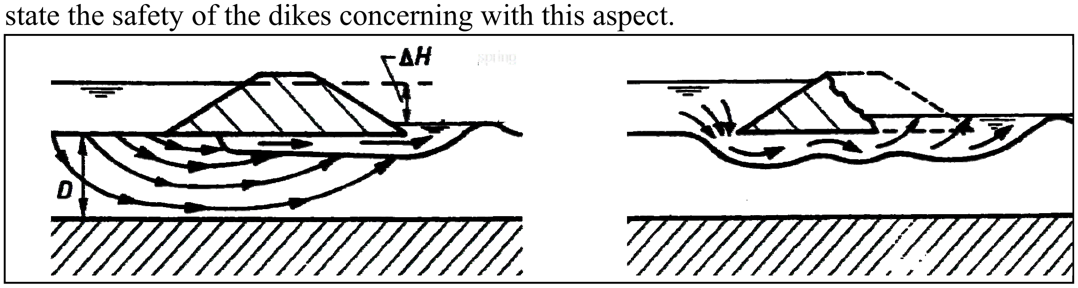


Figure 4.7: Piping mechanism in sand layer underneath the dike.

4.2.2.4 Deformation and settlement of dike's body

Vertical deformation due to reduction in volume of soil which can be considered as settlement. The settlement may be caused by the following mechanisms:

- Consolidation -squeezing out of pore water from a cohesive soil
- Compression - more dense packing of particles due to extra loading
- Migration etc. - loss of material from (within) the bank
- Shrinkage - drying out of saturated cohesive soil
- Loss of apparent cohesion - some sandy soils exhibit apparent cohesion in dry situation while saturated situation this cohesion is lost, which results in settlement.

When building a dike, settlement as the result of soil compressibility should be taken in to account. In this context, a supplementary height allowance should be added so that the dike will come to rest at planned height after settlement. Beside that it is essential to know the anticipated settlement when calculating the amount of soil to be moved.

Settlement and displacement of body and subsoil of Namdinh dikes will be investigated in chapter 5, section 5.2.3.4.

4.2.2.5 Liquefaction and softening

Loss of consistency due to settlement flow is a mechanism in which a water-saturated mass of sand is subjected to a great displacement as a result of softening. Softening of sand with a loose packing is the result of an increase in shear strength, where, owing to a rearrangement of the grain structure (decrease in volume) an increase of the water and air pressure (pore water pressure) occurs in the pores to such a degree that the contact pressure between the individual grains decreases to a certain degree [TAW-1999]. Significantly, liquefaction presents when complete loss of grain to grain contact by the increase in pore water pressure or by shock loading of a loosely compacted granular soil. Consequence loss of effective stress results in a zero shear strength and the soil behaving as a heavy liquid.

For some of dike sections in Namdinh there is a sand foreland in front of the dike. This foreland at some places was settled by sand at low density due to sediment transport processes. Such sand is sensitive to loosening under the fully saturated condition at high water level. Hence it is possible for the occurring of liquefaction of the foreland. This type of liquefaction may be dangerous for the dikes.

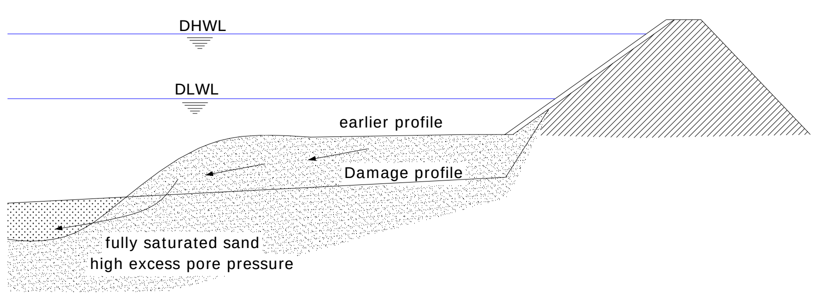


Figure 4.8. Mechanism of possible liquefaction at Namdinh sea dikes

4.2.3 Structural failure modes (revetment)

4.2.3.1 Instability of armour layer.

Instabilities of the armour layer of the revetment is one of the most regular failure in Vietnam. There are some of reasons which caused the failure. First, the thickness of the cover layer is not sufficient to the hydraulic condition. This due to the fact that most of the design for that were still applied by the old method of the year 60s. Second, the construction of the revetment was implemented mostly by hands therefore the quality of the constructions was not good.

Generally, the armour layer is interlocking concrete blocks or rocks. However, in Vietnam it is very difficult to find the big rocks, which have the diameter larger than 30cm. Because the rocks have been exploited by explode and the limited fund leading to not popular in using the concrete blocks. Therefore the suitable rock size is chosen for the cover layer of revetments very limited.

The stability of the armour layer of the existing revetment at Namdinh sea dikes will be checked by methods both in Vietnam codes or Dutch codes. Further analysis see Section 5.2.2

Figure 4.8 show the severe damages of revetment after a storm at Haichinh committee. The cover layer was completely collapsed and the filter layer was washing out. In the Figure 4.9, the failure of revetment occurred at the end of section. That is the transition between protected and non protected dikes at Haithinh committee.

Figure 4.8: Damage of cover layer, the filter layer exposures (Haichinh section)

Figure 4.9: Failure of revetment at transition

4.2.3.2 The filter layers

The filter layers which applied for the existing revetment in Namdinh mostly are granular filters. It includes some layers of sand and gravel. However in the recently years, using geo-textile has been popular in design filter layer in Vietnam.

The failure of the filter layer will cause loss of materials of the dike body and/or decrease of drainage of outflow. Therefore it may lead to the serious damages of the dike after some time.

The stability of the filter also needs to be checked order to ensure the safety working state of the revetments as well as the dikes.

During the site visit there were some failure of the filter layer which caused by the returned flow from inland side after action of waves on dike slope. See Figure 4.9

Figure 4.9 Failure of filter layer at Vanly section

4.2.3.3 Toe foot instabilities

The failure of the toe could occur by two main reasons:

- The scour holes appear in front of the toe due to the taking place of foreshore erosion.
- The sizes of the protection are not sufficient to the hydraulic conditions.

Analysis the stability of the toe by the recent method in order to know about it's working state. Based on that, the suggestion for design of the toe will be given.

In Figure 4.10, the failure of the toe structure leads to the damage of under part of the revetment. This picture was taken at HaiTrieu section.

Figure 4.10: Failure of toe structure leads to damage of revetment (HaiTrieu section)

CHAPTER 5 DETERMINISTIC ASSESSMENT OF THE SAFETY OF NAMDINH SEA DIKES

In this chapter, based on the analysis of possible failure mechanisms in chapter 4, the safety of sea dikes in Namdinh is being estimated by applying Vietnamese and Dutch design codes. The overall safety of the dikes is contributed by the safety of related components. Thus all the failure modes which presented in chapter 4 should be investigated. However, due to the limited data and time, the investigation will be performed for some priority failure mechanisms.

Comparisons of using two design codes will be made. Based on that, some remarks and recommendation on new sea dike design will be carried. In order to find out what will be the differences and what will be the influences in dike design by using Vietnamese codes and Dutch codes, the same boundary conditions will be applied.

5.1 Definition of boundary condition.

5.1.1 Load boundary conditions.

It is obvious that the boundary conditions for sea dikes are induced mainly from water levels and waves. In general these are called natural boundary condition. The quality of the subsoil, important for the geotechnical stability, is sometimes also called a boundary condition. However in these notes the quality of the subsoil is treated as a separate item in the analysis of the limit states, because the quality of the soil is more related to the strengths than to the loads. Important for the construction of a sea dikes are:

- The maximum water levels and the significant height of the waves;
- The magnitude of the water level-difference between the inner side and the outer side of the dikes;
- The magnitude of the water levels difference along the dike (occurrence of currents)
- The time between high and low water;
- The number of times that this water level difference occurs during the lifetime of the construction;
- The rising and failing of the water (in how much time the water-level rises from normal to extreme).

A lot of relevant information for a seawall and sea dike design can be drawn from files and existing maps. In addition to this, a field reconnaissance and a land survey are indispensable, as well as photographic recording of the characteristic points in the area. Special attention should be paid to the position of the beach and or onshore profiles, and the morphology of the area considered (eroding/ accreting coast). The composition of the existing dike body and the geologic structure of the subsoil are also very important. When these data are not available the soil mechanical investigation should be considered (soundings, borings etc.).

Definition of the situations

In order to give the clear comparison of how safe the dikes are at completed construction time and at the moment, in this chapter the solving of all problems will be considered by two situations of the site. These are design situation and current situation.

- *Design situation* is the situation of the site (include the beach level, bathymetry and dimension of the dikes) at the time of implementation of the dikes.

- *Current (present) situation* is the situation of the site which was investigated recently, September, 2003.

5.1.1.1 Design water levels.

The maximum water level is needed to estimate the maximum breaking wave height at the structure, the amount of run-up to be expected, and the required crest elevation of the structure. Minimum expected water levels play in important role in anticipating the amount of toe scour that may occur and the depth to which the armour layer should extend. The following components will contribute to design water level:

a. Astronomical tides. Changes in water level are caused by astronomical tides with an additional possible component due to meteorological factors (wind setup and pressure effects).Periodic tidal levels are published annually by the National Oceanic and Atmospheric Administration.

b. Storm surge (wind setup): Storm surge can be estimated by statistical analysis of historical records, or through the use of numerical models. The numerical models are usually justified only for large projects. Some models can be applied to open coast studies, while others can be used for bays and estuaries where the effects of inundation must be considered.

In addition to that for sea dikes design we also take in to account other types of water fluctuations like, see Pilarczyk. K.W et al, Dikes and revetments, 1998

- Wind setup
- Climatologically variations
- Wind waves
- Squall oscillations (seiches) and gust bumps

c. D.W.L by Dutch actual Design Codes (DDC):

According to Dutch design codes (DDC) the design water level (DWL) is given by:

$$\text{Dutch DWL} = \text{M.S.L} + Z_{tide} + \Delta Z_{wind} + \Delta Z_{gust} + \Delta Z_{rise}$$

In which :

M.S.L : Mean Sea Level. The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings.

Z_{tide} : Tidal level which measure at high water spring (refers to MSL)

ΔZ_{wind} : Increase in water level in front of the dikes due to Wind setup (and storm surge). This is a rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress

ΔZ_{gust}: Rise of water level due to gust bump
ΔZ_{rise} : Rise of water level due to Sea level rise.

Determination of the components: The elevation which will be determined are all relative to CD (chart datum- Standard elevation system in Vietnam)

M.S.L : At the study area, M.S.L is +1.92m relative to Vietnam-HD coordinates (The " Zero" value is at the lowest annual water level of sea surface). It corresponds to +0.0 meter original land coordinates.

Tidal level: Z_{tide}=+2.29 m +MSL, is the averaged highest tidal range for the location according to annual publication of Vietnam Marine Hydro-meteorological Center.

Wind setup (Storm surge):

- This can be estimated by 2% frequency of occurrence from exceedance curve of surge level which based on the measurement during 19 years period. By doing so the increase in water level due to wind setup is 1.0 m

- On the other hand wind setup can also be determined from the equation based on the definition is the difference in still-water levels on the windward and the leeward sides of a body of water caused by wind stresses on the surface of the water (Pilarczyk K.W et al, Dikes & Revetments-1998).

$$\Delta S = \sqrt{\frac{2C_w(\rho_{air}/\rho_w)U^2F}{g}\cos\phi + h^2} - h$$

In which: C_w friction coefficient, value of $0.8\text{x}10^{-3}$ to $3\text{x}10^{-3}$
ρ_{air} : density of air, 1.25 kg/m^3
ρ_w : density of sea water, 1031 kg/m^3
F : fetch length, 150 km
h : water depth at calculated position, 3m corresponding to M.S.L
Φ : Angle between wind direction and normal of coastline, it is 30^0
U : wind velocity corresponds to storm with wind strength of 9 Beaufort

Applying the equation by given parameters, end up with **wind setup of 1.2m.**

Gust bump: this is the increase in water level due to wind surge in normal conditions. In this case it is neglected.

Sea level rise: for many years it has been known that the sea is level rising. In historical time this sea level rise was generally moderate. The sea level rise here is considered as the rise of water level itself plus the changes of the level of the land. The relative sea level rise can determined from long time series of water level observations. In Vietnam there was not any long term record for these observations. However the predictions of sea level rise were made based on the subsidence of the land in Vietnam and the rise in water level of South China Sea. Here the applied sea level rise is **ΔZ_{rise} =20cm/century**

Seiches: Not taken in to account for Namdinh coastline because there are not any heavy navigation activities.

Since related component are known, the DWL can be determined as in Table 5.0:

d. D.W.L by Vietnamese actual Design Codes (V.D.C):

DWL is highest water level which may occur, and corresponding to the design frequency. It can be determined from the exceedance curve of water level observations. The observed water level included tidal water level, seiches, gust … however it does not take in to account sea level rise.

$$\text{Vietnam DWL} = \text{M.S.L} + Z_{tide} + \Delta Z_{wind} + \Delta Z_{gust}$$

Based on 5% of design frequency and exceedance curve over 19 years of tidal level observation, the corresponded sea water level takes value of +2.29 (m +MSL).

Wind setup (storm surges) is based on design frequency and latitude of the location. Here Namdinh is around 20 northern latitude and 2 % of design frequency so wind setup is 1.0 m. Hence, design water level can be determined in Table 5.0:

Table 5.0: Determination of design water level at Namdinh sea dikes

Notation	*Component*	*Unit*	*Vietnam*	*Dutch*
M.S.L	Mean sea level (+CD)	m	0	0
Z_{tide}	Tidal high level (+MSL)	m	2.29	2.29
ΔZ_{wind}	Wind Setup	m	1.0	1.2
ΔZ_{gust}	Gust bump	m	0	0
$\Delta Z_{seiches}$	Seiches	m	0	0
ΔZ_{rise}	Sea level Rise (50 years)	m	not included	0.1
DWL	**Design water level (+MSL)**	m	3.29	3.6

The DWL which was applied for previous design of existing dikes was at 3.29 m (+MSL), see ‘‘Design documents of Namdinh sea dikes, 1996’’

Conclusion:

The old value of DWL for existing dikes in Namdinh is exactly the same new one (computed value of DWL) which was applied by recent design codes for sea dikes in Vietnam. Both are at 3.29 m (+MSL).

The design water level is determined by Dutch design codes for Namdinh coast is at 3.60 m (+MSL). The difference in DWL of 30cm is considerable when applied different codes. The difference are due to, firstly, different approach of determination of wind setup level, secondly, VDC do not account for the rise of sea water level.

5.1.1.2 Design wave heights.

a. Design wave height definition:

Wave heights and periods should be chosen to produce the most critical combination of forces on a structure with due to consideration of the economic life, structural integrity and hazard for events that may exceed the design conditions. Wave characteristics may be based on the analysis of wave gauge record, visual observation of wave action, published wave hindcasts or the maximum breaking wave at the site (depth limited waves). Wave characteristics derived from such method may be for deep water location and must be transformed to the structure site using refraction and diffraction techniques. Wave analyses may have to be performed for extreme high and low design water levels and for one or more intermediate levels in order to determine the critical design conditions.

b. Wave data along Namdinh coast

There is not sufficient observations of wave data at the locations for design purpose. However the significant wave high offshore Namdinh coast can be estimated by using wind data of design storm. The wind data for the study area is measured at Bach Long Vi Island, the location of the island see Figure 5.1.

Prediction of offshore waves by using wind data is based on the method of Breschneider 1952 (then developed by Sverdrup and Munk). The background of the method is that the wave height and wave period is a function of the wind-velocity, the fetch length and the water depth.

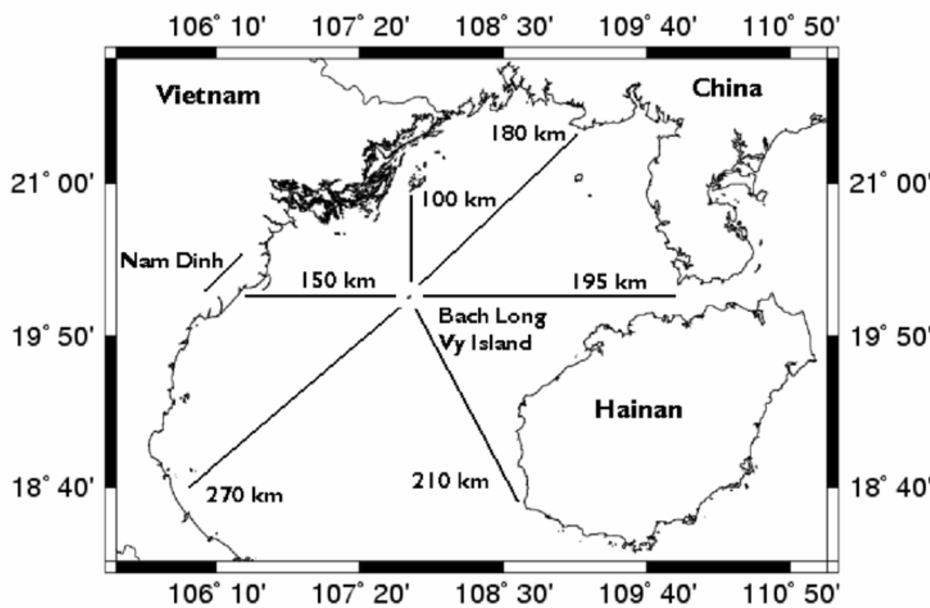


Figure 5.1: Tonkin Gulf and location of Bach Long Vy Island [Vu et al,2003]

For Namdinh sea dike with dike grade of 2, therefore the design storm is of 9 Beaufort. This storm corresponds to wind speed of U_{10}=22 m/s (measured at Bach Long Vi island); the fetch length is about 150km, then the predicted significant wave height is 4.0 meter and significant wave period is 8.5 seconds (see Table 5.1).

Table 5.1: Estimation of wave height by using wind data

Parameters	*Unit*	*Value*
U_{10} (Storm of 9 Bf)	m/s	21.8
F	km	150
g	m/s2	9.81
F*	-	3096.33
H*	-	0.08334
T*	-	3.75446
H_s	m	4.0
T_m	m	8.5

c. Design waves at the toe of the dikes:

- A wave propagating from deep water to shallow water will undergo refraction, shoaling and breaking at some point which depended on wave steepness and relative water depth. Wave breaking along the foreshore will limit the maximum significant wave height at the dikes. The transformation of significant wave height from deep water to position of the toe results in design wave height for the dikes.

- Design wave height for Namdinh sea dikes can be obtained by two ways:

- Transferring the significant wave height, this was predicted by using wind data of design storm, from offshore to the position of the toe, taking in to account shoaling, refraction and breaking.

- Using depth-limited wave height at the position of the toe. The local wave height in front of the dikes can be determined by the relation to the local water depth, d. The so called depth-limited wave height for Namdinh condition can be roughly estimated as $H_{s,max}$=0.55d.

The design wave height by applying both methods is summarized in Table 5.2.

Table 5.2: The design wave heights for considered situations and conditions

Situations	*Local water depth (m)*		*Depth limited wave height (m)*		*Transformed wave height (m)*	
	Design Situation	*Present Situation*	*Design Situation*	*Present Situation*	*Design Situation*	*Present Situation*
at Vietnam DWL (+3.30 m)	3.8	4.6	2.1	2.6	2.1	2.5
at Dutch DWL (+3.60 m)	4.1	4.9	2.3	2.7	2.3	2.7

- For the present situation of the site, which was investigated recently in Dec 2003, the design wave height is 2.6m when applied depth limited estimation; and 2.5m when transforming significant wave height from deep water to the position of the toe. To be in safe side, the depth limited wave height is suggested to use for all the later calculations.

- The design wave height at present situation is 0.5 meters larger than that at design situation. This value is remarkable in design point of view.

- For previous design of Namdinh sea dikes, the design wave height Hs was applied by 2.0 meters and there was not any explanation of what kind of wave height was used for design and how it was determined. It was underestimated when using this wave height if comparing to either depth limited wave height or transformed wave height.

- Mean wave period is 8.5 seconds, based on the wave prediction by using wind data.

- The approached angle of incident waves in deep water can be considered identically with the dominant wind angle. For the researched location, the dominant wind angle to the normal of coastline is about 30^0. Therefore the angle of incident waves in deep water is 30^0. The angle of wave attack at the toe of the dike is obtained by refraction transferring wave from deep water to that location. Yields with $\beta=24.8^0$

5.1.2 Strength boundary conditions.

The quality of subsoil, the properties of dike's body soil and also the geometrical design parameters of dike's components are considered strength boundaries. All the calculations in this chapter will deal with the boundary conditions of the design situation and existing situation.

5.2 Safety of the dikes by applying Vietnam and Dutch design codes.

5.2.1 Impact of wave run-up, wave overtopping and crest level to the related failures

5.2.1.1 Investigation of Wave run-up and wave overtopping computation

a. Run-up:

This is the maximum elevation of wave uprush above still-water level (Figure 5.2). Wave uprush consists of two components: super elevation of the mean water level due to wave action (setup) and fluctuations about that mean (*swash*). The upper limit of runup is an important parameter for determining the crest height of the dikes.

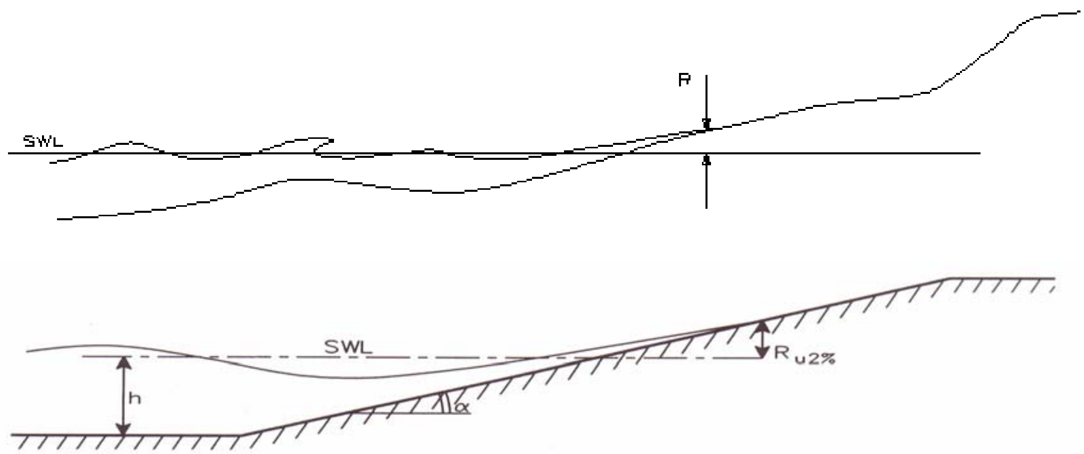


Figure 5.2: Definition sketch for wave run-up and wave run-up on a slope of a dike

Wave run-up at a dike is often indicated by $R_{2\%}$ (or $\Delta Z_{2\%}$), see Figure 5.2. This is the run-up level, vertically measured with respect to the still water level, which is exceeded by 2% of incoming waves.

Wave run-up calculation in VietNam

Present design practice in Vietnam various formulae are used for prediction of wave run-up. There is no recommendation of which formula is the most actual and should be used. In the reviewed design of sea dikes the following two Russian formulae were mostly used. The oldest formula which was applied for calculation of wave run up in Vietnam given by:

$$\Delta Z = 3.8Hs * \frac{\cos\beta}{\cot an\alpha} \qquad (1)$$

And the second oldest one:

$$\Delta Z = 2Kn * \frac{Hs}{\cot an\alpha} * \sqrt[3]{\frac{Ls}{Hs}} \qquad (2)$$

Where Hs is local wave height; α is outer slope angle; β is angle of wave front approaching the coast; Ls is local wave length; and K_n is reduction coefficient due to roughness of a slope.

When using these formulae, there were not any indication on estimation of wave height and no indication of how far the formulae used is appropriate for condition under consideration.

In the formulae (1) no effect of the slope roughness is included. It looks to be similar to the Dutch formulae:

$$\Delta Z = 8 * Hs * \tan\alpha \qquad (3)$$

However, the numerical coefficient in formula (1) is about a half of the Dutch formula, which has been proved to work satisfactorily for smooth slopes and wave steepness of about 4 %. This means that the formula (1) with coefficient 3.8 should only be used for riprap slopes where run-up is reduced by factor 0.5. Using this formula for revetments or pitched stone and 2% wave run-up the numerical coefficient should be increased instead of 3.8.

The second formula (2) is a more general formula because of including effect of slope roughness Kn and wave steepness Hs/Ls. It was derived from Russian Design Code and had been used in Vietnam since 1990. However it was not clear about the range of validation of the components in the formulae and how to determine the components when applying.

Recently the latest formulae for calculation of wave run-up was introduced in 2002 (refer to Chinese codes)

$$\Delta Z_{p\%} = \frac{K_\Delta . K_w K_p}{\sqrt{1+m^2}} \sqrt{Hs.Ls} \qquad (4)$$

This formulae was written in Vietnam codes for design of sea dikes which was published in 2002. In which :

- ΔZ : wave run-up with exceedance probability of p%
- K_Δ : reduction factor for slope roughness
- K_w : coefficient of wind effect
- K_p : run-up coefficient in function of percentage of wave exceedance
- m = cotanα with α is outer slope angle.
- H_s : significant wave height near the toe of the dikes.
- Ls : Wave length at the toe of the dike (m)

This is considered the most general formulae for determination of wave run-up in Vietnam for the time being. It includes the influences of roughness of slopes, the steepness of slope, wave steepness and effect of wind in front of the structures via an empirical coefficient K_w. Moreover there is an indication on estimation of applied wave characteristics. According to that, the applied wave height and wave length are estimated at the toe of the dike. Furthermore it also said that the formulae appropriates for condition of slope ranging from 1.5 to 5. However it is not clear explanation of Kw in Vietnamese Code. This empirical coefficient, by indication, can be determined from empirical Table of value based on the term of $W/\sqrt{g.h}$ (Table D-2, Vietnamese Codes) while W is wind speed in front of the dikes. In fact K_w stands for the effect of wind, according to Chinese Code.

The wave run-up at Namdinh sea dikes is found out by applying the above formulae for design and existing situations, at Vietnam DWL combines with design wave height. The results are tabulated in Table 5.3.

In these calculations the *reduction factor of slope roughness is 0.55*, determined for riprap, random dumped stones in 2 layers on a granular sub-layer as filter. The wind effect coefficient K_w is in relation of $W/\sqrt{g.h}$ with wind speed of 22 m/s.

Table 5.3 : Wave run-up level by difference formulae

Parameters	*Unit*	*Riprap*		*Block*		*Con.Slab*	
		(1)	*(2)*	*(1)*	*(2)*	*(1)*	*(2)*
Water depth	m	3.80	4.60	3.80	4.60	3.80	4.60
Wave height (at the toe)	m	2.10	2.60	2.10	2.60	2.10	2.60
Wave period(mean)	s	8.50	8.50	8.50	8.50	8.50	8.50
Deep water wave length Lo=1.56T^2	m	162.30	162.30	162.30	162.30	162.30	162.30
Local wave length Ls = T*(g*h)^1/2	m	62.28	68.52	62.28	68.52	62.28	68.52
Angle of wave front	degree	30.00	30.00	30.00	30.00	30.00	30.00
Foreshore slope	-	0.01	0.01	0.01	0.01	0.01	0.01
Steepness of outer slope	-	4.00	4.00	4.00	4.00	4.00	4.00
Kn- reduction factor for slope roughness	-	0.55	0.55	0.75	0.75	0.90	0.90
Wind speed in front of the dikes W	m/s	21.80	21.80	21.80	21.80	21.80	21.80
Kw- empirical coefficient	-	1.00	1.00	1.00	1.00	1.00	1.00
Hs/h	-	0.55	0.57	0.55	0.57	0.55	0.57
Kp - transformed coefficient	-	1.61	1.61	1.61	1.61	1.61	1.61
Wave run-up by applying formulae 1	m	1.73	2.14	1.73	2.14	1.73	2.14
Wave run-up by applying formulae 2	m	1.79	2.13	2.44	2.90	2.93	3.48
Wave run-up by applying formulae 3	m	4.20	5.20	4.20	5.20	4.20	5.20
Wave run-up by applying formulae 4	m	2.46	2.87	3.35	3.91	4.02	4.69

Column (1) : deign situation *Column (2) : Present situation*

Recent Dutch formula for wave run up: The general design formula that can he applied for wave run-up on dikes is given by :

$$\frac{R_{u2\%}}{H_s} = 1.6\gamma_f \gamma_b \gamma_\beta \xi_{op} \qquad (5)$$

In which :

$R_{u2\%}$ is 2% run-up level above the still water line, exceeded by 2% of the number of incoming waves.

H_s is significant wave height near the toe of the structure

ξ_{op} is breaker parameter

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\frac{2\pi H_s}{gT_p^2}}}$$

ξ_{eq} is equivalent breaker parameter for a slope with a berm

$$\xi_{eq} = \gamma_b \xi_{op}$$

T_p : peak period of the wave spectrum

γ_b : reduction factor for a berm

γ_f : reduction factor for slope roughness

γ_β : reduction factor for oblique wave attack

The formula is valid for the range 0.5 < ξ_{eq}< 4 or 5. The relative wave run-up depends on the breaker parameter ξ_{op} and on four reduction factors, namely for the influence of a shallow foreshore (breaking waves at shallow water), roughness of the slope, oblique wave attack and for a berm. The influence of a berm is accounted for by an equivalent slope gradient expressed in ξ_{eq}. In the case of no berm, then $\xi_{eq} = 1.\xi_{op}$ applies.

Applying recent general Dutch formulae (5) 2%-wave run-up is determined for the same 2 above situations. The results are in Table 5.4.

The other reduction factors can determined according to J.W.van der Meer and J.P.F Janssen 1994:

- The reduction factor for slope roughness γ_f =0.55 determined for armour rock in 2 layers thick; γ_f=0.80 for block revetment; and γ_f=0.90 for concrete slab revetment.

- The reduction factor for oblique wave attack γ_β=1-0.0022|β| for 0≤|β|≤80

Table 5.4 : Wave run-up by Dutch formula (J.W.van der Meer, 1994)

Parameters	*Unit*	*Riprap*		*Block*		*Con.Slab*	
		(1)	*(2)*	*(1)*	*(2)*	*(1)*	*(2)*
Water depth	m	3.80	4.60	3.80	4.60	3.80	4.60
Wave height (at the toe)	m	2.10	2.60	2.10	2.60	2.10	2.60
Wave period(mean)	s	9	9	9	9	9	9
Deep water wave length Lo=1.56T^2	m	162.3	162.3	162.3	162.3	162.3	162.3
Angle of wave front	degree	23	23	23	23	23	23
Foreshore slope	-	0.01	0.01	0.01	0.01	0.01	0.01
Steepness of outer slope	-	4	4	4	4	4	4
h/Hs	-	1.81	1.77	1.81	1.77	1.81	1.77
Breaker parameter	-	2.20	1.98	2.20	1.98	2.20	1.98
Reduction factor for a berm	-	1	1	1	1	1	1
Reduction factor for slope roughness	-	0.55	0.55	0.80	0.80	0.90	0.90
Reduction factor for oblique wave attack	-	0.95	0.95	0.95	0.95	0.95	0.95
Relative wave un-up	-	1.43	1.41	2.08	2.05	2.34	2.30
Wave run-up .	m	3.01	3.66	4.37	5.32	4.92	5.98

Column (1) : deign situation *Column (2) : Present situation*

It is clear that the run-up on a dike will be strongly influenced by the slope angle (m = cotanα) and the roughness and permeability of a slope (reduction factor γ_r or K_Δ). A high roughness and a high permeability of the revetment provide a high reduction of run-up. In Namdinh the popular type of revetment is riprap, this is random dumped stones in 2 layers on a granular sublayer of filter layer. This provides a 40% reduction (reduction factor γ_r or K_Δ = 0.60). For some sections which have revetment covered by pitched stones and blocks will provide the reduction factor of about 0.85 to 0.90, this means that there is 15 % to 10% reduction of wave run-up. The concrete slabs were also applied for some sections which can consider providing practically no reduction.

When applying the actual Dutch formulae, the deepwater wave length is defined as $L_{op} = 1.56T_p^2$ (T in sec, L in meter). The local wave length (L) due to the shoaling effect will be reduced approximately to $L_s = T^*(g.h)^{1/2}$, where T is wave period in deep water and h is local water depth. For long shallow foreshore there will be a combination of waves transformed from the deepwater and the locally developed waves. This will affect the shape of the wave spectrum. The existing run-up formulations are based on wave steepness defined by a local wave height H_s and the deepwater wave length L_{op}. For conditions with long shallow foreshore and resulting depth-limited wave height it provides for dike slopes 1 on 3 and 1 on 4, a breakwater-index ξ_{op} larger than 2 to 2.5. In such case the wave run-up becomes independent of wave period and can be defined as the maximum value:

$$\frac{R_{u2\%}}{H_s} = 3.2\gamma_f\gamma_\beta \tag{6}$$

The latest Dutch formulae for wave run-up, introduced by TAW 2002:

$$\frac{Z_{2\%}}{H_{m0}} = 1.77\gamma_b\gamma_f\gamma_\beta\xi_o = 1.77\gamma_f\gamma_\beta\xi_{eq} \quad \text{for } 0.5\le\xi_{eq}\le1.8 \tag{7a}$$

$$\frac{Z_{2\%}}{H_{m0}} = \gamma_f\gamma_\beta(4.3-1.6/\sqrt{\xi_o}) \quad \text{for } 1.8\le\xi_{eq}\le8 \text{ to } 10 \tag{7b}$$

Where:

$z_{2\%}$ is 2% run-up level above the still water line, exceeded by 2% of the number of incoming waves.

H_{mo} is significant wave height at the toe of the structure

ξ_o is breaker parameter

$$\xi_o = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H_{mo}}{gT_{m-1,0}^2}}}$$

ξ_{eq} is equivalent breaker parameter for a slope with a berm

$$\xi_{eq} = \gamma_b\xi_o$$

$T_{m-1,0} = m_{-1}/m_o$ is spectrum wave period, can determined in relation of $T_p=1.1T_{m-1,0}$

m_o; m_{-1} : Zero and first negative moment of spectrum

T_p : peak period of the wave spectrum

γ_b : reduction factor for a berm

γ_f : reduction factor for slope roughness

γ_β : reduction factor for oblique wave attack

In this formula the influence factor for a shallow foreshore has been removed. However for an influence of shallow foreshore it is advised to look for a relationship of $H_{2\%}/H_{mo}$ (in addition to the reduction of wave height itself instead of addition to the reduction of wave run-up level). In the case of Namdinh sea dike, the depth limited wave height is used. By using formula (7) for calculation of wave run-up, the results are shown in Table 5.5

Table 5.5 : Wave run-up by Dutch formula (J.W.van der Meer, 2002)

Parameters	*Unit*	*Riprap*		*Block*		*Con.Slab*	
		(1)	*(2)*	*(1)*	*(2)*	*(1)*	*(2)*
Water depth	m	3.80	4.60	3.80	4.60	3.80	4.60
Wave height (at the toe)	m	2.10	2.60	2.10	2.60	2.10	2.60
Wave period(mean)	s	8.5	8.5	8.5	8.5	8.5	8.5
Peak wave period	s	10	10	10	10	10	10
Spectrum wave period	s	9.3	9.3	9.3	9.3	9.3	9.3
Deep water wave length Lo=1.56T^2	m	134.1	134.1	134.1	134.1	134.1	134.1
Angle of wave front	degree	26	26	26	26	26	26
Foreshore slope	-	0.01	0.01	0.01	0.01	0.01	0.01
Steepness of outer slope	-	4	4	4	4	4	4
h/Hs	-	0.25	0.31	0.25	0.31	0.25	0.31
Breaker parameter	-	2.00	1.80	2.00	1.80	2.00	1.80
Reduction factor for a berm	-	1	1	1	1	1	1
Reduction factor for slope roughness	-	0.55	0.55	0.80	0.80	0.90	0.90
Reduction factor for oblique wave attack	-	0.94	0.94	0.94	0.94	0.94	0.94
reduction factor for a shallow foreshore	-	0.577	0.591	0.577	0.591	0.577	0.591
Relative wave un-up	-	1.64	1.61	2.39	2.35	2.69	2.64
Wave run-up .	m	3.45	4.19	5.02	6.10	5.65	6.86

Column (1) : deign situation *Column (2) : Present situation*

The comparison between applying actual formula for wave run-up on slope of 1 on 4 at Namdinh sea dikes with various kinds of revetments is shown in Table 5.6

Table 5.6 : Comparison of wave run-up on various revetments

Wave run-up	*for design situation*			*for existing situation*		
	Riprap	*Blocks*	*Con. slap*	*Riprap*	*Blocks*	*Con. slap*
applied design value for constructed dikes	2.0	2.2	2.5	-	-	-
VN new design code (formula No 4)	2.46	3.35	4.02	2.87	3.91	4.69
actual Dutch formulae (formula No 5)	3.01	4.37	4.92	3.66	5.32	5.98
Latest Dutch formula (formula No 7)	3.45	5.02	5.65	4.19	6.10	6.86

b. Overtopping

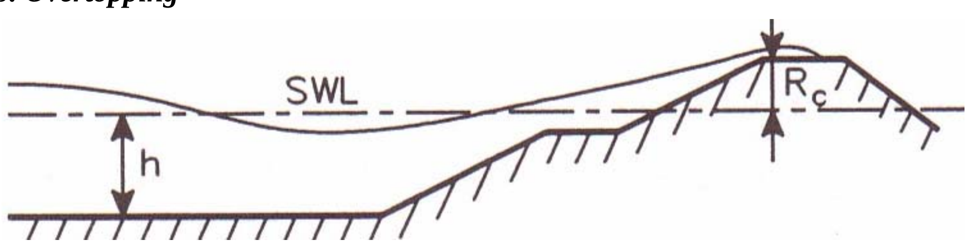


Figure 5.3: Wave overtopping at a dike.

For a dike with wave overtopping, the crest height is lower than the run-up levels of the highest waves. The parameter to be considered here is the free crest height R_c, see above Figure. This is the difference in level between SWL (D.W.L) and the crest height. Wave overtopping is mostly given as an average discharge q per unit width, usually in m^3/s per m or in litter/s per.

For the time being there is not any concerns with wave overtopping at sea dikes during design processes. Even in the latest design code for sea dikes which were published in 2002, the wave overtopping is not included. So there is not any criteria of wave overtopping when design of the dikes.

In the Netherlands, the Dutch Guideline on river dykes (TAW, 1989) indicates that for relatively heavy seas and with wave heights of up to a few meters the 2%-wave run-up criterion yields an overtopping discharge of the order of 1 l/s per m. It becomes 0.1 l/s per m with lower waves such as those occurring in rivers. An acceptable overtopping of 1 l/s per m in the river area, instead of 2%-wave run-up, can lead to a reduction of the freeboard of the dike. In Dutch Guidelines it is assumed that the following average overtopping rates are allowable for the inner slope:

- 0.1 l/s per m for sandy soil with a poor turf
- 1.0 l/s per m for clayey soil with relatively good grass
- 10 l/s per m with a clay protective layer and grass according to the standards for an outer slope or with a revetment construction

The former formulae on wave overtopping was developed by Van der Meer and Janssen 1995 (see TAW 1998) and improved by Van der Meer 2002 (see TAW 2002). The recent formulae for wave overtopping on dikes is as following:

$$\frac{q}{\sqrt{gH_{mo}^3}} = \frac{0.06}{\sqrt{\tan\alpha}}\gamma_b\xi_o e^{-4.7\frac{R_c}{H_s}\frac{1}{\xi_o\gamma_b\gamma_f\gamma_\beta\gamma_v}} \qquad (7)$$

where q : averaged overtopping rate (m^3/s per meter width);

R_c : crest free board (m)

γ_v : reduction factor due to a vertical wall on a slope

Applying for given case on Namdinh sea dike, the results of overtopping calculation are shown in Table 5.7.

Table 5.7: Required freeboard by wave overtopping condition.

Parameters	*Unit*	*Riprap*		*Block*		*Con.Slab*	
		(1)	*(2)*	*(1)*	*(2)*	*(1)*	*(2)*
Critical overtopped discharge q	l/s/m	1.0	1.0	1.0	1.0	1.0	1.0
Required crest free board R_c	m	2.72	3.03	3.96	4.41	4.46	4.96

The height of free board for existing dike at this moment at Namdinh is 2.20 m.

The results of wave run-up and wave overtopping at Namdinh sea dikes for various kind of slope protection are summarized in the Table 5.8

Table 5.8 Wave run-up and overtopping at Namdinh sea dikes with Vietnam DWL

Situation	*DWL*	*Local Depth*	*Design wave*		*type of slope protect.*	*wave run-up height by different formula (m)*				*q*	*freeboard R_c by overtop. criteria*
			Hs	*Tm*		*Old VN code*	*new VN code*	*Dutch 1995*	*Dutch 2002*		
	(m)	*(m)*	*(m)*	*(s)*		*Form. (1)*	*Form. (4)*	*Form. (5)*	*Form. (7)*	*l/s/m*	*(m)*
(1)	(2)	(3)	(4)	(5)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Existing dike	3.3	4.00	2.00	-	riprap	2.00	-	-	-	-	2.20
					block	2.20				-	2.20
Determ. for design situation	3.3	3.80	2.10	8.50	riprap	1.73	2.46	3.01	3.45	1.0	2.72
					block	1.73	3.35	4.37	5.02	1.0	3.96
					concrete	1.73	4.02	4.92	5.65	1.0	4.46
Determ. for present situation	3.3	4.60	2.60	8.50	riprap	2.14	2.87	3.66	4.19	1.0	3.03
					block	2.14	3.91	5.32	6.10	1.0	4.41
					concrete	2.14	4.69	5.98	6.86	1.0	4.96

The results show that the wave run-up which was applied for previous design is too small. The underestimation of wave run up for existing dikes is about from 1 to 1.5 m at design situation and from 1 to 2.5m at present situation when comparing to different formulae.

The formula (1) did not depend on the roughness of the slope. It should be applied only for rock revetment with about 55% reduction of wave run up height. However for the old design of sea dikes in Vietnam, this formula was used for also bock and concrete revetment.

Formula (4) is the latest one for wave run-up calculation. However the result of wave run-up height is still underestimated when comparing to the recent Dutch formula.

The freeboard according to the overtopping criteria is higher than that when applying wave run-up criteria given by recent Vietnam design code (formula 4)

5.2.1.2 Investigation of design crest level

The design crest level of existing sea dikes in Namdinh was applied at +5.5m for the some of the most important section. The crest of other parts was applied at level of +5.0 m. And this crest level does not change when applying different type of slope protection.

In this section, the design crest level of Namdinh dike will be determined for the case of riprap protection of outer slope, which providing the lowest required crest level. After that the comparison between the crest level of existing dike and the calculated values will be made.

a. Design crest's level according to Vietnam design codes:

At present the design crest level in Vietnam is determined by the latest design code, given by the following formulae:

$$Z_c = D.W.L + H_{sl} + a \qquad (8)$$

DWL : is design water level which determined from above section.

H_{sl} : is height of wave run up. For sea dike, it is determined as height of 2% wave run-up ($Z_{2\%}$)

a : is safety free board, it was given by based on grade of the dike. For Namdinh sea dikes with grade of 2, the safety free board of 0.4 m was recommended in the latest design codes.

It is clear that the crest level of a sea dike in Vietnam is based only on the criteria of wave run-up. There is not any indication dealing with overtopping for determination of crest level.

By applying formulae (8), the crest levels of the dike with riprap revetment are in Table 5.9

Table 5.9: Crest level of the dike by Vietnam Design Codes - Run up criteria
(applied a=0.4m, wave run-up on riprap slope of 1 on 4)

Parameter	*unit*	*for design situation*			*for existing situation*		
		form. (4)	*form. (5)*	*form. (7)*	*form. (4)*	*form. (5)*	*form. (7)*
DWL(+MSL) by Vietnam code	m	3.3	3.3	3.3	3.3	3.3	3.3
wave run-up 2%	m	2.5	3.0	3.5	2.9	3.7	4.2
Safety free board	m	0.4	0.4	0.4	0.4	0.4	0.4
Computed design crest level	m	6.2	6.7	7.2	6.6	7.4	7.9

b. Design crest level according to Dutch design codes:

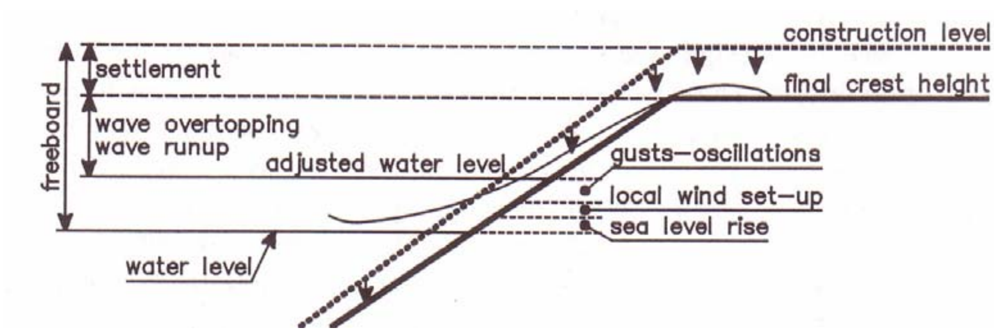


Figure 5.4: Components contribute to design crest level of the dikes

The formulae for determination of design crest level of a dikes in Netherlands is:

Z_c***=DWL+free board+Expected settlement of subsoil and/or dike body.***

The free board is chosen in order to meet two criteria

- criteria of 2% wave run-up (i)
- criteria of overtopping (overtopped discharge q=1.0 l/s/m) (ii)

Table 5.10: Design crest level of the dikes , according to overtopped criteria
(wave overtopping for riprap slope of 1 on 4)

Parameters	*for design situation*	*for existing situation*
Design water level	3.30	3.30
Expected settlement (3% of the dike height)	0.15	0.15
Free board by wave overtopping	2.70	3.00
Crest level	6.15	6.45

Table 5.11: Design crest level of the dikes, according wave run-up criteria

Parameter	*unit*	*for design situation*			*for existing situation*		
		form. (4)	*form. (5)*	*form. (7)*	*form. (4)*	*form. (5)*	*form. (7)*
DWL(+MSL)	m	3.3	3.3	3.3	3.3	3.3	3.3
wave run-up 2%	m	2.5	3.0	3.5	2.9	3.7	4.2
expected settlement	m	0.15	0.15	0.15	0.15	0.15	0.15
Crest level	m	5.9	6.5	6.9	6.3	7.1	7.6

c. Summary

The crest level of existing dike in Namdinh is not high enough to ensure safety criteria of wave run-up and wave overtopping for both current situation and design situation. The reasons are:

- Underestimation of wave run up level by applied the old formula and by applying the formula in un-proper way.

- Did not take in to account the overtopping criteria when determined design crests level.

- The difference in water depths in front of the dikes between design situation and current situation leads to the differences in wave heights. The water depth was getting deeper from design situation to the current situation. As result wave heights near the toe of the dike will increase. Consequently the wave run-up as well as overtopping of the current situation is also larger than that at design situation. Therefore the crest level of existing dike is no longer sufficient enough for the current boundary condition.

- The design water level by applying Vietnam design code is 30 cm less than that by applying Dutch design code. This difference caused underestimation of dike height at lease (or more than) 30 cm. For the same situation the rise of DWL caused directly rise in crest level of the dike and, moreover, increase in water depth in front of the dikes. Due to that the design parameters includes wave height, wave run-up, wave overtopping will get bigger. Therefore the height of the dikes needs to be higher.

- Design crest level by Vietnam design code which given by formula (8) does not include the expected settlement. However it contains the safety free board ‘a’. The value of ‘a’ given by recommended table in the design code which based on grade of dike. The explanation of ‘a’ value should also be given in the design code.

- Design crest level of Namdinh sea dikes was applied at +5.5m for the some of the most important section (with riprap revetment). However the required crest level of the dike for present situation is +6.45 according to Dutch overtopping criteria (formula 8); +6.3m by the latest Vietnamese run-up criteria (formula 4); And up to +7.6m by the latest Dutch run-up criteria (formula 7).

c. Recommendation

- Determination of DWL should be given more clearly and should take in to account also sea level rise, expected settlement.

- More attentions should be paid when selecting the design wave height. Depth limited wave height is a good reference to check any method of design wave height estimation.

- The wave run-up formulae in the design codes should be investigated and given the range for application. It can only be applied for design of new dike when it is calibrated.

- More advance wave run-up formulae and their validation should be added in the new design codes

- The criteria of wave overtopping for design crest level must be added in the new design codes.

- Design crest level by Vietnam design code which given by formula (8) does not include the expected settlement. However it contains the safety free board ‘a’. The value of ‘a’ given by recommended table in the design code which based on grade of dike. The explanation of ‘a’ value should also be given in the design code.

5.2.1.3 Failure mechanisms related to insufficient design crest level

The failures which did and which may occur due to underestimation of wave run-up and wave overtopping and due to insufficient of design crest level:

- Heave damage of crest and crown wall (see pictures)
- Erosion of inner slope
- Inundation at lee side due to too much of overtopped sea water.
- Damage of upper part of revetment due to the return flow of overtopped sea water where the lee side of the dikes is housing area.
- Washing material at outer side where the dike body was not well protected by cover layers.

5.2.2 Design of revetments and safety investigation for related failure modes

5.2.2.1 General information

a. Functional requirement of revetments

By definition, a revetment is a slope protection designed to protect and stabilize a slope that may be subject to action by water currents and waves - To fulfil this function, the following aspects have to be taken under consideration in the design process:

- Stability: top-layer, sub-layer, subsoil, foundation.
- Flexibility: the layers of revetment follow the settlement without influencing the stability.
- Durability: top-layer, asphalt, concrete, geotextile, cables.
- Possibility of inspection of failure (monitoring of damage)
- Easy placement and repair (local damage).
- Low cost (construction/maintenance)
- Overall safety (primary or secondary defence, geometry of foreshore, etc.).
- Additional functional requirements

b. Various types of revetment and their critical modes of failures

Type of revetment: There are numerous types of revetments. Most of which have been tested in a systematic way by physical models in small-scale and large-scale wave flumes, see CUR 1995.

- Rip-rap or uniform rock revetment is applied in many projects. An extensive experimental research program has been carried out to obtain design rules for these types of slope revetments (Van der Meer and Pilarczyk, 1984, 1986).
- Concrete armour units are applied for groins, breakwaters and for slope protection. A large number of model investigations have been carried out in order to optimize the design of concrete armour layers.
- Regularly placed natural stones or concrete blocks are used for seawalls and dikes. A number of basic research programs is in progress in order to study the phenomena which occur on a sloping revetment of regularly placed elements.
- Other types of revetments have been tested recently, some of these tests being commissioned by the manufacturers. These include: bags or sausages filled with sand, gravel, cement, rock, etc. gravel, gabions (wire mesh containers filled with relatively coarse material), asphalt, and grass on a clay layer, geotextiles.

Choice of revetment (Krystian W.Pilarczyk et al, 1990- Coastal protection).

It is obvious that there are very many possible combinations that can lead to a large number of possible constructions based on the classification of revetments. This does not simplify the choice of a revetment. Besides, the choice of the main revetment construction has its own repercussions for the transitions and the other parts of the dike, and the execution and maintenance method. To make choice out of various and in a certain situation possible alternatives criteria for judging need to be formulated (functional, technical and financial) with the help of the demands that are made. Even the best design may fail as a result of poor workmanship and bad management. Hence, the aspects which are concerned with construction and with management and maintenance should also be involved in this stage. Because all the various criteria have not been defined equally well and do not play an equally prominent part in the definite choice, subjective experiences and/or prejudice can be decisive. It seems to be wise to make the choice in a group so that the subjective aspect can play the least possible part. For the different aspect weighing factors can be made so that a more objective choice might be possible. This problem has been actually treated and mentioned in Dutch design codes for revetment (TAW, 1988).

The situation of revetment design for sea dikes in Vietnam:

- Lack of theoretical knowledge and practical experiences
- Poor investment, lack of annual maintenances
- Low quality of construction due to lack of proper managements in construction phases

5.2.2.2 Namdinh revetments and applied boundary conditions.

Along sea dike there are there types of slope protection. Riprap revetments with random dumped stone by 2 layers are applied for mainly length of Nadinh sea dikes. Beside that concrete interlocking block revetments were presented at some new sections which were implemented under trial projects of coastal protection. The rest part of Namdinh sea dikes are some protected by stone-paved revetments, concrete plates and some other protected by grass mats. In this section the safety investigations are made for two popular types of slope protection along Namdinh sea dikes, these are riprap and block revetments. Present design cross section of Namdinh revetments is in Figure 5.5, 5.6:

Riprap-block revetment:

- The cross section looks like Figure 5.5
- From level +3.5 to the level at the toe (-0.5m) was protected by concrete slaps with dimensions of 0.40x0.40x0.28m.
- From level + 3.5 to level of crest was protected by rip-rap rock with thickness of 0.30m.
- Toe of revetment: one line of cylindrical concrete block, diameter of 100 cm and length of 150 cm, rock filled inside

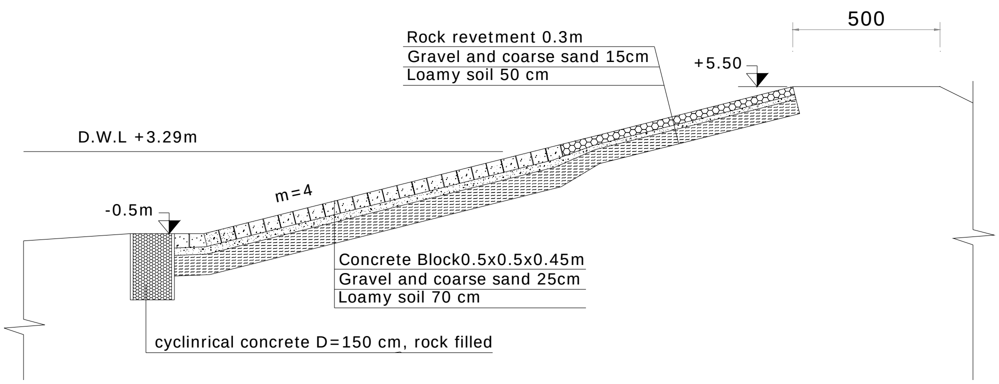


Figure 5.5: Mixed riprap block revetment- Applied at Namdinh

Block revetment:

- The cross section looks like Figure 5.6
- From crest level +5.50 to the level at the toe (-0.5m) was protected by hexagonal concrete block with thickness of 30 cm.
- Toe of revetment: one line of cylindrical concrete block, diameter of 100 cm and length of 150 cm, rock filled inside

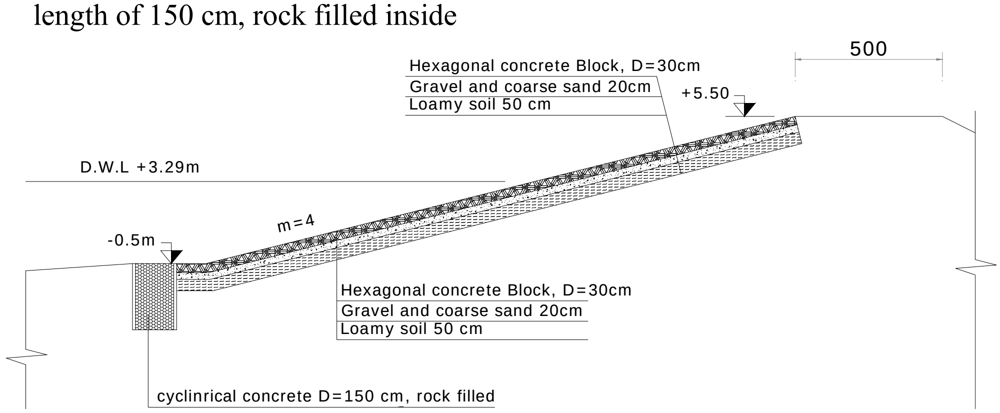


Figure 5.6: Hexagonal concrete block revetment- Applied at Namdinh

The safety investigations are based on boundary conditions of design situation and present situation as indicated in Table 5.12.

Table 5.12. Common boundary condition for Namdinh revetments

Parameter	*unit*	*Design situation*	*Present situation*
High DWL	m	+3.3	+3.3
Low water level	m	+0.1	+0.1
Toe level	m	-0.5	-0.5
Sea bed level in front of the toe	m	-0.5	-1.3
Mean period of design waves	s	8.5	8.5
Design wave height	m	2.1	2.6

For the given types of revetments and physical boundary condition, the safety of each component will be carried out by applying Vietnam actual design codes. Then the same problems will be investigated by Dutch actual design code for revetment.

5.2.2.3 Safety of slope protection of the dikes by applying Vietnamese Design Codes

a. Armour layer of revetment

a1. Required dimension of stones/block, size of rock

At present situation of sea dikes in Vietnam, Russian formulae and Hudson formulae are used. All there formulae are originally developed for riprap and rubble mound structures. The outcome of these formulae is required weight and thickness (size) of element.

The required weight of block for old existing revetment (design in 1992 and constructed in 1993) in Namdinh which applied Russian formula was W_{50}=375 kg for slope of 1 on 4. The used formula was not written down however the indication said that it was Sa-Ba-Nop formula.

Later on since 1996, the two other Russian formulae were used for design of revetment.

- The required thickness of the block d_m was determined by Sankin (Russian) formula:

$$d_m = 1.7 * \frac{\gamma_n}{\gamma_d - \gamma_n} K_1 H_s \qquad (9)$$

where K1 is sloping coefficient, which depends on slope of revetment. For slope of 1 on 4, given K1=0.17 (Design document of Namdinh sea dike, 1997).

- The required weight of slope protection was calculated by V.X Saitan (Russian) formula.

$$Q_d = 0.0142 * \gamma_d \frac{\gamma_n}{\gamma_d - \gamma_n} H^3{}_s * F(\alpha)$$

where F(α) is slope influence coefficient, For slope of 1 on 4, given F(α) =1.16 (Design document of Namdinh sea dike, 1997).

- Applying the formula for Namdinh revetments, wave height Hs=2.10 m, slope 1 on 4

For concrete block revetment(Δ=1.30),

=> d_m =0.42 m, and the chosen value of 0.45m.

=> Q_d=276 kg, and the chosen value of 280 kg

For riprap revetment (natural stone, Δ=1.55),

=> d_{m50} =0.39 m, and the chosen value of 0.40m.

=> Q_{d50}=245 kg, and the chosen value of 250kg

In the latest design code for the revetment: the Hudson formula, Chinese formula and Pilarczyk's formulae were recommended to use.

Hudson formula: The applied form in Vietnam design code for revetment was given by:

$$G = \frac{\gamma_b H_{sd}^3}{K_D \left(\frac{\gamma_s - \gamma_n}{\gamma_s} \right)^3 .\cot an\alpha} \qquad (10)$$

In which, the parameter was defined:

G is minimum weight of cover element

γ_s, γ_n are unit weight of used material and water

H_{SD} is Design wave height, given by significant wave height $H_{1/3}$.

K_D is stable factor, depends on the type of cover element, given by Table 5.13

Table 5.13: Stability factor according to VDC

Type of element	*Type of placement*	K_D
Stones	Random dumped in 2 layer	3
Stones	Free paved placement	4
Concrete block	Independent placement	3.5
Concrete block	Slim-interlocking	5 to 6

(See Vietnam design codes for sea dikes, 2002)

The required averaged diameter (D_{n50}) (size of element) can be derived from (10) when consider the weight of element equals to D^3_{n50} (linear relations). For one-layer system, D_{n50} can also be considered as the required thickness of armour layer.

$$\frac{H_s}{D_{n50}\Delta} = \left(K_D \cot g\alpha\right)^3 \qquad (10a)$$

$$D_{n50} = \left(\frac{G_{50}}{\gamma_s} \right)^{1/3} \qquad (10b)$$

As the result by applying the formulae (10a) and (10b), the Figure 5.7 shows the stability condition corresponding to 3 types of slope protections. For given conditions of Namdinh revetment, the outcome is summarized in Table 5.14.

Table 5.14:Rock size and required weight for the revetments.

Parameters	Unit	Design Situation			Current Situation		
		riprap	Paved-Stones	Concrete block	riprap	Paved-Stones	Concrete block
Hs	m	2.10			2.60		
W_{min}	kG	525.6	394.2	674.7	997.6	748.2	1280.4
D_{50}	m	0.58	0.53	0.66	0.72	0.66	0.81

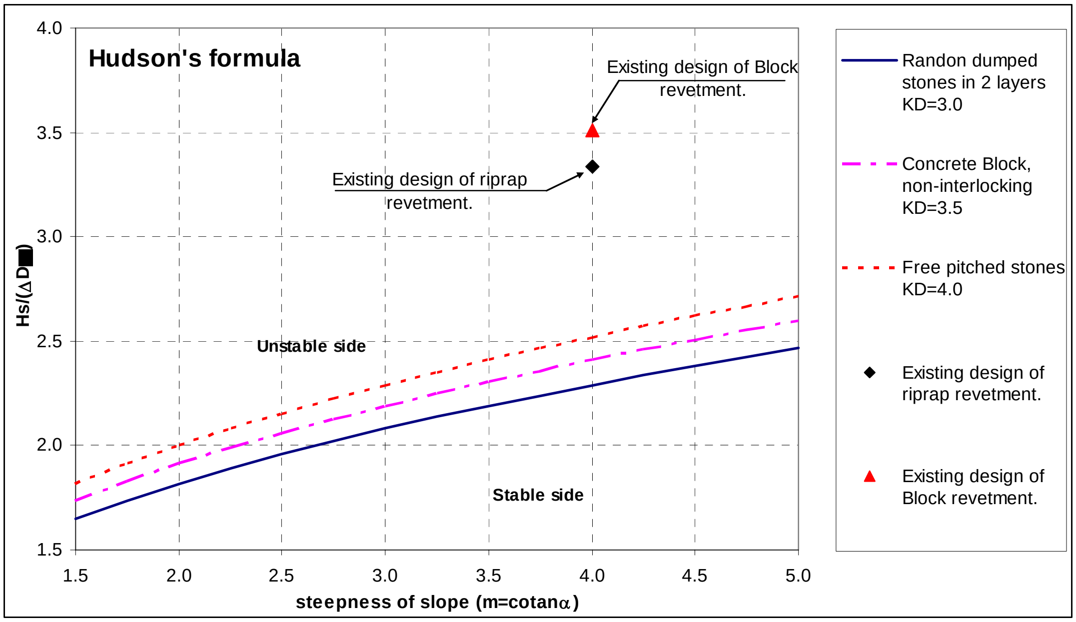


Figure 5.7: Stability of revetments according to Hudson formula

As the result from the above Table, it is clear that:

- For the existing riprap revetments, the rock size which was used of 30 to 50 cm (approximate 70 to 325 kG) is much smaller than the requirement for both design and present situation.

- For existing block revetment, the thickness of block was 0.45m (block dimension: 0.5x0.5x0.45, weight approximates 250 kG) is also too small compares to the requirement thickness of 0.66meters at design situation; 0.81 meters at present situation.

Comment on using Hudson's formula in Vietnamese codes:

- The Hudson formula under the form of (10) is dimensional form. It expresses that the weight (mass) always representative for stability. However the weight (mass) do not distinguishes the difference between the flat and more round stones. Therefore the stability should be expressed by dimensionless parameter. To avoid this it is better to represent Hudson's formula in linear, dimensionless form as (10a) and (10b).
- Applying Hudson's formula for n-layer system, the total required thickness is not equal to D_{50} any more. It should be calculated by: $t=n*c*D_{n50}$ where c is layer thickness coefficient (see Shore Protection Manual, 1995).
- In case of two layers system the averaged weight of rock or block must be between 0.75G to 1.25G and at least 50% of the stone/block must be heavier than the average.
- The Hudson's formula does not include the effect of wave period, therefore, for more precise calculation the more actual formulae should be used (e.g Van der Meer's formula, and/or Pilarczyk's formula).

a.2 Thickness of armour layer

In present design codes of sea dike in Vietnam, there are three formulae which introduced for calculation the thickness of armour layer depend of type of elements for slope protection (type of armour layers).

For rock revetment

- *First Chinese formula:*

$$\delta_s = 0.266\frac{\gamma_w}{\gamma_s - \gamma_w}\sqrt[3]{\frac{L_s}{H_s}}\frac{H_s}{\sqrt{m}} = 0.266\frac{H_s}{\Delta\sqrt{m}}\sqrt[3]{\frac{1}{S_{H_s}}} \qquad (11)$$

$$\delta_s = 0.266\frac{H_s}{\Delta\sqrt{m}}\sqrt[3]{\frac{1}{S_{H_s}}}$$

where: δ_s is the thickness of top layer (1 layer, m)

H_s, L_s : local significant wave height and correspondent wave length.

$H_s = H_{s4\%}$ if $h/L_s \geq 0.125$

$H_s = H_{s1/3}$ if $h/L_s < 0.125$

γ_s, γ_s : volumetric weight (or mass) of stones/block and water (kN/m^3)

The design code indicated that formula (11) is applied for ***free paved stone revetment (riprap).*** However, the original form of formula (11) was more general as follow (Chinese design code-GB50286-98):

$$t(orD) = K_1\frac{\gamma_w}{\gamma_s - \gamma_w}\sqrt[3]{\frac{L_s}{H_s}}\frac{H_s}{\sqrt{m}} \text{ or, re-written to: } \frac{H_s}{D\Delta} = \frac{(tg\alpha)^{1/6}}{K_1}\xi_s^{-2/3} \qquad (11a)$$

K_1 : coefficient; for free pitched stones 0.266; for cement grouted (masonary) stones 0.225, use when the interspaces are washed-in with granular material. The formula (11a) gives the value as shown in Figure 5.8. As can be seen from the Figure, the existing design points lie in unstable side. This means that both the existing free-pitched stones and existing cement grouted stone revetments may not be stable under the design condition. The required values for design and current situations are tabulated in Table 5.15

Table 5.15: The required size of stone for slope protection by formula (11a)

Parameter	*Hs*	*ξop*	*free pitched stones*		*cement grouted stones*	
			Hs/ΔD	*D*	*Hs/ΔD*	*D*
applied value of existing design	2.1	2.15	3.01	0.45	3.51	0.45
required for design situation	2.1	2.20	1.765	0.77	2.087	0.65
required for present situation	2.6	1.98	1.895	0.89	2.241	0.75

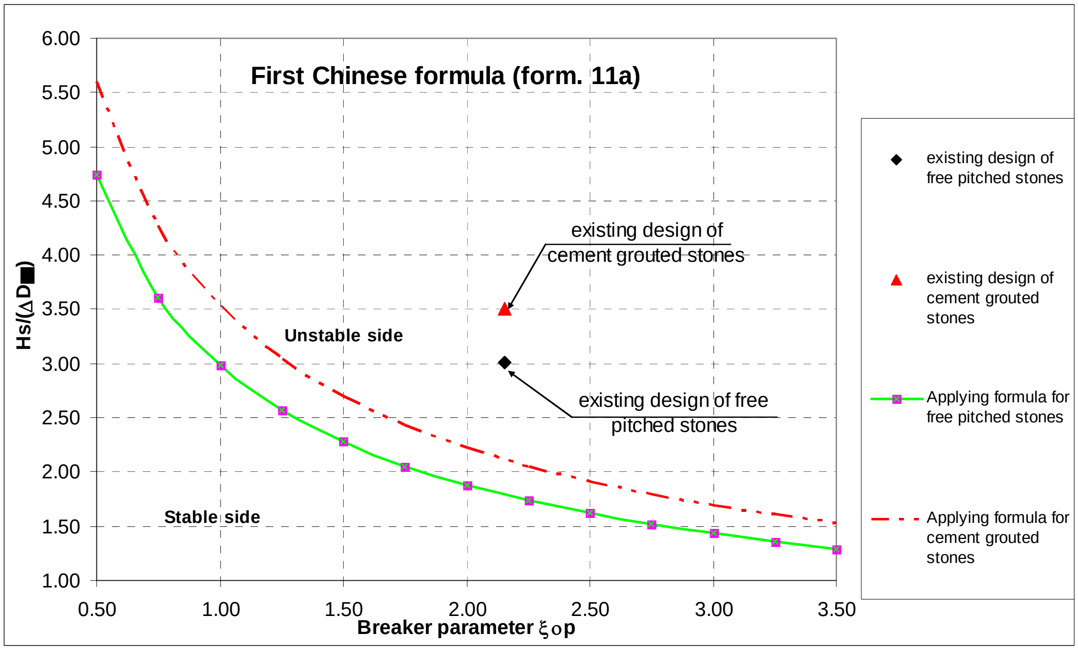


Figure 5.8: Stability of revetments by first Chinese formula (11a)

Comment on using of first Chinese formula in Vietnamese Code

- The first Chinese formula, which is used in actual Vietnamese design code, was introduced by a fixed form as formula (11) with K_1=0.266. Therefore the formula can only be applied for free/pitched stones. Moreover the form (11) is and under dimensional form so it is difficult to compare with the more actual formulae which usually under the dimensionless form. The more general form of formula (11) should be re-introduced as formula (11a).

- The definition of cement grouted stones is not clear. In case of all stones are grouted then it should be treated as slab because there are not any interspaces for being as blocks. Therefore more information should be introduced when using this formula.

For block revetment: For armour layer of concrete-slaps (block revetment), the required thickness is considered the maximum value when applying the two following formulae:

- ***Second Chinnese formulae***

$$\delta_c = \eta . H_s . \sqrt{\frac{1}{\Delta} . \frac{L_s}{l_t . m}} \qquad (12)$$

In Vietnam design code, this formula is indicated for design of protection of slope by concrete slabs (large, flat concrete blocks). Where the parameter are defined:

δ_c is the required thickness of concrete slab

H_s is averaged wave height of 1% highest waves.

l_t is the length of slab side which perpendiculars to water line

η is a coefficient which depends on how the slabs are placed, η = 0.0075 for non-grouting, paved slaps; η = 0.10 for slabs of lower side with grouting, upper side with non-grouting paved. This definition of η is not clear, and there is one mistake when said "η = 0.0075", instead of η=0.075 (it could be a typing mistake!!!).

In Chinese design code η was defined as slab-coefficient: η = 0.0075 with joined slaps; and η = 0.10 with not connected slabs (large blocks). However there was not any clear explanation of applicable type of slabs and slabs placement by using this formula (e.g. what kind of connection between slabs, size of interspaces, free or filled-in…)
This formula can be re-written to a more general form as:

$$\frac{H_s}{D\Delta} = \frac{\sqrt{l_t \tan\alpha}}{1.5.\eta.\sqrt{\Delta H_s}} \frac{1}{\xi_{op}} \qquad (12a)$$

It should be noted that a slab was defined as a large block with $l_t/D>5$ and $B/H_s>1$. Therefore the formula (12) as well as (12a) is not applicable for checking stability of Namdinh revetments. The given values of formula (12a) are in Figure 5.12.

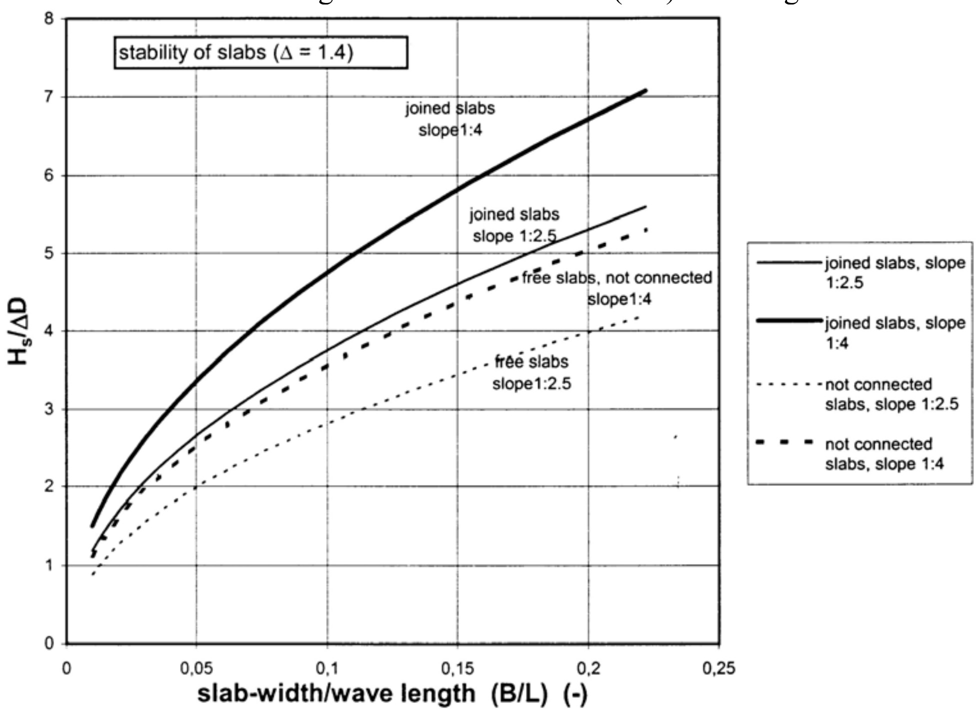


Figure 5.9: Stability of concrete revetment by second Chinese formula (12/12a)

- ***Pilarczyk, K.W formulae:***

The formula of Pilarczyk which used in Vietnamese design code was written as:

$$\delta_c = \frac{H_s}{\varphi}\frac{1}{\Delta}\xi^{\frac{2}{3}} \quad \text{(or rewritten as : } \frac{H_s}{D_{50}\Delta} = \frac{\varphi}{\xi_{op}^{\ 2/3}}\text{)} \qquad (13)$$

where:

H_s is significant wave height (averaged wave height of one third highest waves)
ξ is breaker parameter given by:

$$\xi = \frac{\tan\alpha}{\sqrt{\dfrac{H_s}{L_s}}}$$

The application of formula (13) is only for concrete slabs protection which uses of the following stability factor φ depends on shape and placed type of slabs/large block.

φ=4 to 4.5 for placed blocks,

φ=5 for placed blocks on geo-textile and good (smooth surface) clay,

φ=6 for pitched columns (e.g Basalton) washed in by coarse material,

φ=8 for interlocked blocks on properly designed sub-layers and subsoil.

In addition the formula can be applied for rock armour layer with φ=3 (for pitched natural stone/riprap).

Given value of the formula for slope of 1 over 4 are shown in Figure 5.13.

Three existing design points of Namdinh revetments are lies in unstable area of each curve, respectively. The existing design of riprap revetment compares to line label φ=3 (Δ=1.55); the line label φ=4 or 4.5 (Δ=1.33) is stable criteria for placed block revetment; and line label φ=8 (Δ=1.33) is stable criteria for interlock block revetment.

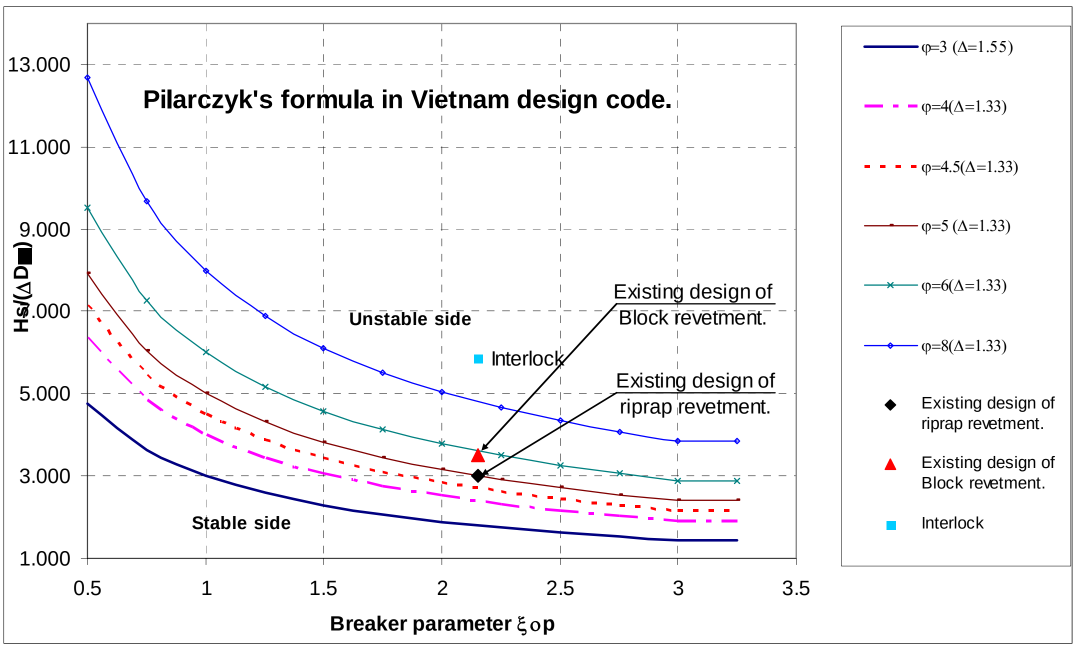


Figure 5.9: Applied Pilarczyk's formula in Vietnamese design code.

Applying these formulae the required thickness/size of slabs/blocks for given cases of Namdinh revetments are in Table 5.16 and Figure 5.9.

Table 5.16: Required size of stones and thickness of block for slope protection by Pilarczyk's formula (13)

	Hs	*ξop*	*Riprap Δ=1.55, φ=3*		*Placed Block Δ=1.33, φ=4.5*		*Interlock (Tac-178) Δ=1.33, φ=8*	
			Hs/ΔD	*D(m)*	*Hs/ΔD*	*D(m)*	*Hs/ΔD*	*D(m)*
applied value of existing design	*2.1*	2.15	*3.01*	0.45	*3.51*	0.45	*5.85*	0.25
required for design situation	*2.1*	2.20	*1.77*	0.76	*2.66*	0.51	*4.73*	0.29
required for present situation	*2.6*	1.98	*1.91*	0.88	*2.86*	0.59	*5.08*	0.33

Comment on using of Pilarczyk's formula:

The original form of Pilarczyk's formula is:

$$\frac{H_s}{D_{50}\Delta} = \psi_u \frac{\varphi}{\xi_{op}{}^{b}} \qquad (\Rightarrow \text{therefore } D = \frac{1}{\psi_u}\frac{H_s}{\varphi}\frac{1}{\Delta}\xi_{op}{}^{b}) \qquad (13a)$$

This formula can be applied for various type of slope protection by proper selection of it's parameters (see also Pilarczyk et al, Dikes & Revetments, 1995).

- The formula was introduced under the very specific form with fixed parameters compare to the original form. As indication in the recent Vietnamese design codes, the formula can only be applied for calculation of concrete slabs for slope protection. However, when applying formula of Pilarczyk for concrete slabs system, the parameter ψ_u lies from 3.0 to 4.0 instead of 1.0 as applied form in Vietnam design code.
- When determining: $\xi = \dfrac{\tan\alpha}{\sqrt{\dfrac{H_s}{L_s}}}$, in Vietnamese design code the definition of wave length is not clear. It must be used the deep water wave length L_{op} which corresponds to peak period T_{op}. Then ξ_{op} should be introduced instead of ξ.

- When applying formula of Pilarczyk for riprap, it is note that the parameters $\boldsymbol{\psi_u}$ lies from 1.0 to 1.33 and coefficient ***b*** is ½ instead of 2/3.

- In the case of natural pitched stones/riprap it is found that the Pilarczyk's criteria is lies between first Chinese criteria (formula 11a) for free pitched stones and cement grouted stones, see Figure 5.10.

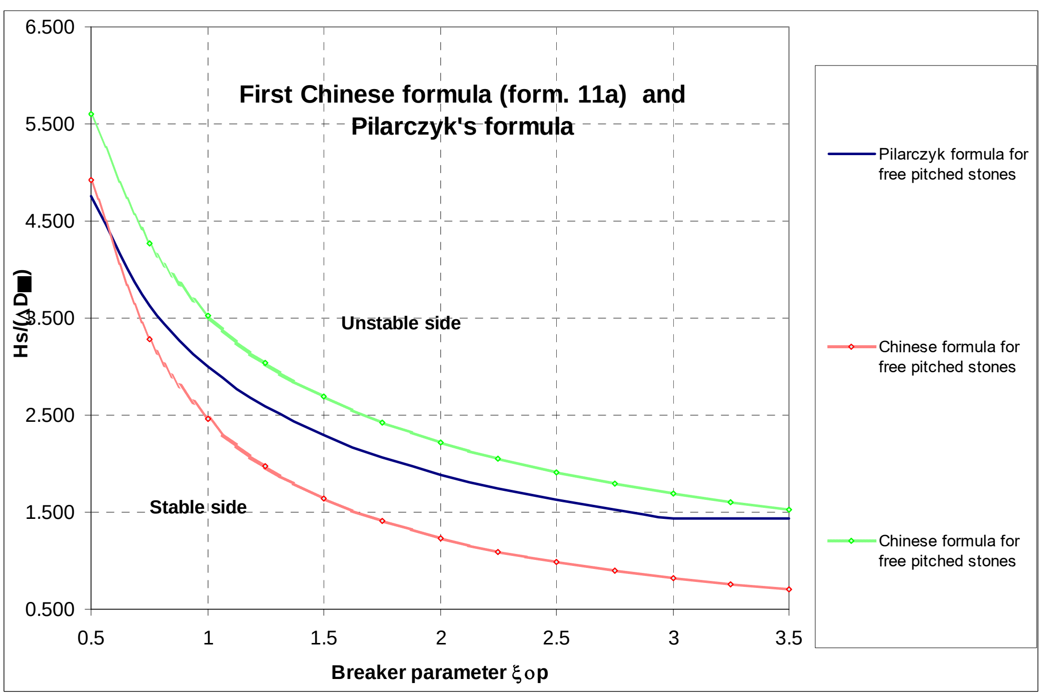


Figure 5.10: Comparison between Pilarczyk's and first Chinese formula.

b. Toe protection

Determinations of dimensional design parameters of toe protected structure are based the empirical method given in the code. There are not any given formulae in present design document and also in the latest design codes.

The size of rock which applied for toe protection was determined based on comparison between maximum velocity of currents and critical velocity of applied rock.

The maximum velocity of wave induced currents is as followed:

$$V_{\max} = \frac{\pi . H_s}{\sqrt{\frac{\pi . L_s}{g} . \sinh \frac{4\pi . h}{L_s}}}$$

Where Hs is significant wave height at the toe of the dike; Ls is local wave length; h is local water depth.

For each value of V_{max} the size of rock can be determined the given value as follow:

V_{max} (m/s)	*2.0*	*3.0*	*4.0*	*5.0*
Weight of rock (kG)	*40*	*80*	*140*	*200*

Applying for given boundary condition of Namdinh Dikes, the required weight of toe protection is given in the below table. The required weight of rock for toe protection for Namdinh revetments is 52 kG at design situation and is 80 kG for current situation while the averaged value of rock applied for existing design of Namdinh is W_{50}= 55 kG (rock size of 15 to 35 cm). Therefore the existing of toe protection for revetments in Namdinh is not sufficient under design condition for current situation.

Situation	*h(m)*	*Ls*	*V_{max}(m/s)*	*Required rock weight (kg) W_{50}*
Applied value for existing design				55 (45 to 70)
Design situation (Hs=2.1m)	3.8	52.0	2.3	52 (45 to 65)
Present situation (Hs=2.6m)	4.6	57.1	3.0	80 (65 to 100)

Moreover, it is necessary to include the loading impact of local currents (e.g. tidal currents) into the toe's calculation. The suggested formulae can be used such as Jsbash's (1935) and Pilarczyk's (1990) for determination of stone stability again currents.

5.2.2.4 Safety of slope protection of the dikes by applying Dutch Design Codes

a. Rock revetment:

For actual design of rock revetments in Netherlands, Van de Meer's formula is used. The formula was written for deep water and shallow water condition.

Deep water condition:

For plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5} \qquad (14)$$

For surging waves:

$$\frac{H_s}{\Delta D_{n50}} = 1.0P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\sqrt{\cot an\alpha}\,\xi_m^{-0.5} \qquad (15)$$

Shallow water condition:

For plunging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.7P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5} \qquad (14b)$$

For surging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 1.4P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\sqrt{\cot an\alpha}\,\xi_m^{-0.5} \qquad (15b)$$

Where:

P: Notional permeability factor, for Namdinh revetment, the armour layer lies on granular filter layer, underneath is clay layer, so P should take value of 0.1

S: Damage level, the physical description S is that the number of squares with a side of D_{50} that fit into the erosion area or S is number of cubic stone with a side of D_{50} eroded within a D_{50}-wide strip of the structure. In the case of slopes of 1 on 4 the initial damage level taken as S=3 (see Jan Van de Meer, 1993).

N: Number of waves, it is recommended to use 7500 waves in order to obtain that the situation more or less reaches equilibrium (to be in economical side).

The given values of these equations are shown in Figure 5.11.

In the case of Namdinh revetments, the shallow foreshore is presented. Therefore form of 'Shallow water condition' is recommended to be used. In shallow water condition, instead of using Hs Van der Meer recommended to use $H_{2\%}$. However for the case of Namdinh condition, there is not any reliable statistical wave data therefore the depth limited wave height is used. Assuming that in this case $H_{2\%}=H_s=$ Depth limited wave height. The required rock size by applying Van der Meer's formula is in Table 5.17.

Another alternative for calculation of free pitched stones revetment is using Pilarczyk's formula (formula 13a, with ψ_u=1.0, φ=3, b=1/2). The given result was presented in previous section, see also Table 5.16. Figure 5.9 gives the comparison of using Pilarczyk's and Van der Meer's formulae for rocks revetment. For the case of shallow foreshore they give the same trend however Van der Meer formula lies in more safe-side.

Table 5.17: The required size of rock for slope protection by Van der Meer's and Pilarczyk's formulae

	Hs	*ξop*	*Pilarczyk for Riprap* $\Delta=1.55$, $b=1/2$, $\varphi=3$		*Van de Meer for rocks* $p=0.1$, $S=2$, $N=7500$	
			Hs/ΔD	*D(m)*	*Hs/ΔD*	*D(m)*
applied value of existing design	2.1	2.15	3.01	0.45	3.01	0.45
required for design situation	2.1	2.20	2.02	0.67	1.85	0.76
required for present situation	2.6	1.98	2.13	0.79	1.92	0.86

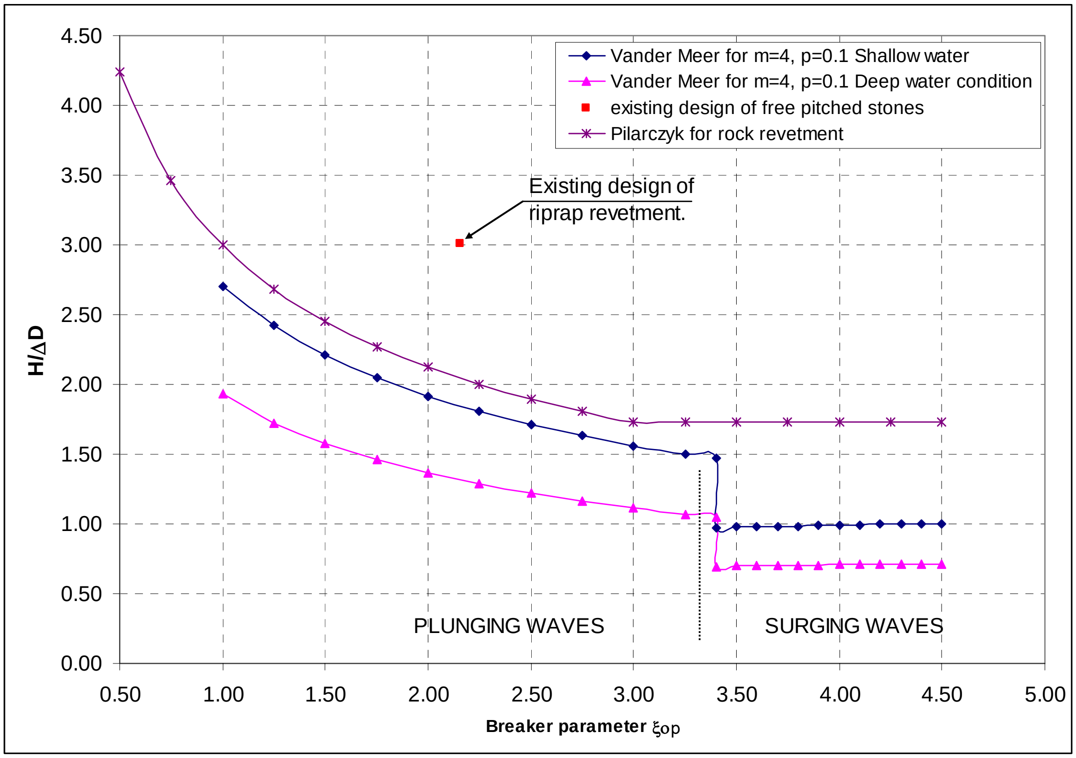


Figure 5.11: Van der Meer's and Pilarczyk's formulae for rock revetment

The computed required rock size is too much exceeding the applied value for existing riprap revetments (applied rock size of 30cm to 50cm , approximates 70 to 325 kG).Therefore the riprap revetment in Namdinh were (or will be) instability under design condition.

b. Concrete slabs and Blocks revetment:

The Pilarczyk formula (Coastal Protection, 1990) can be used for stability calculation of placed block revetments.

$$\frac{H_s}{\Delta_m D} = \psi_u \phi \frac{\cos\alpha}{\xi_z^b} \qquad (16)$$

In which:

ψ_u: System-determined stability upgrading factor. For concrete block, place on granular filter layer, ψ_u = 1.5 to 2.0 (see Table 7.1, p.247 Coastal Protection, 1990)

ϕ: stability factor (stability function for incipient of motion).

b exponent related to the interaction process between waves and revetment type. For place block revetment b=2/3

Apply the formula Namdinh revetments, yielding:

For design situation: H_s=2.2m => the required thickness of block is D_{50}=0.62 m (W_{50}=374 kG). For current situation: H_s=2.6m => required size of stone D_{50}=0.70 m (W_{50}=418 kG)

These results all agree that for existing revetments which covered by concrete (blocks) with dimension of 0.5x0.5x.0.45(appr 250 kG) are no longer stable under design condition. It needs to be upgraded by bigger blocks.

c. Toe protection

In most cases the armour layer on the seaside near the bottom is protected by a supporting toe. If the rock in the toe has the same dimensions as the armour, the toe will be stable. In most cases, however, one wants to reduce the rock size in the toe. The simplest relationship is assumed if the stability number $H/\Delta D_{50}$ is related to the relative depth h_t/h, where h_t is the depth of the toe below still-water level and h is the water depth just in front of the toe. CICIA/CUR (1991) and Van der Meer (1993) gave this relationship with a small number of tests from Delft Hydraulics (DH) and the Danish Hydraulic Institute (DFR). Later Gerding (1993) conducted small scale model tests especially on toe stability, see also Van der Meer et al. (1995).

One of the conclusions of the research was that wave steepness had no influence on stability. Van der Meer et al. (1995) gave an improved formula with respect to toe stability, where the toe depth was given as h_t/D_{n50}. As D_{n50} appeared also in the stability number H_s/D_{n50} it was found later on that for low toe structures unrealistic required toe diameters could be calculated. Therefore, Gerding's work has been re-analyzed. Figure 5.12 gives all his data with a design formula. This formula is almost similar to the original simple formula of Van der Meer (1993), but based on more data points. The formula on toe stability can be written as follows

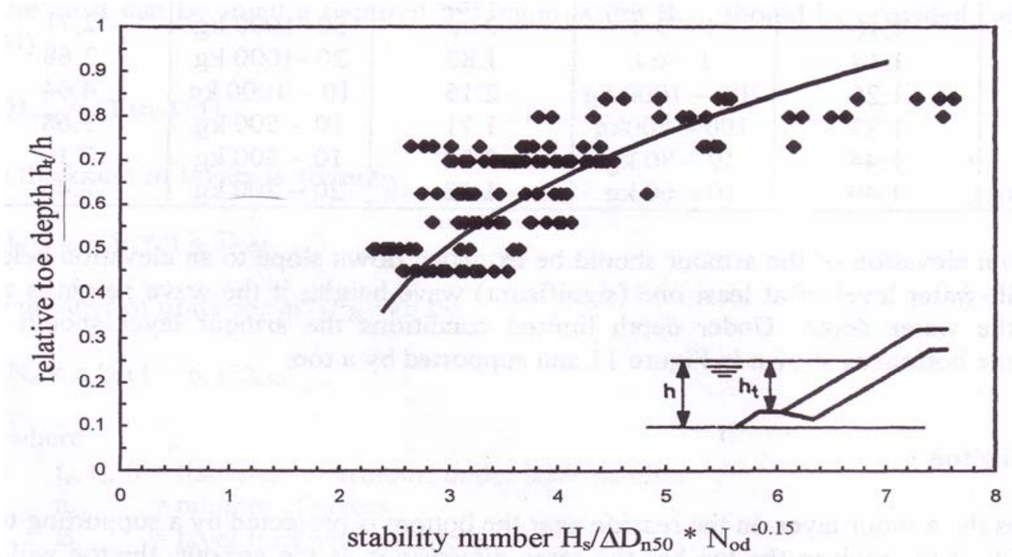


Figure 5.12: Observation data supported to Van der Meer formula(17)

$$\frac{H_s}{\Delta D_{n50}} * N_{od}^{-0.15} = 2 + 6.2(\frac{h_t}{h})^{2.7} \qquad (17)$$

The formula can be used in the range: $0.4 < h_t/h < 0.9$ and $3 < ht/D_{n50} < 25$.
The results from calculation of toe protection for Namdinh revetments show in Table 5.18

Table 5.18: required rock size for toe protection

Parameter	*Unit*	*Situations*	
		Design	*Existing*
Hs	m	2.10	2.60
h	m	3.80	4.60
h_t	m	3.30	3.80
h_t/h	-	0.79	0.74
Δ	-	1.55	1.55
N_{od}	-	2.00	2.00
$H_s/\Delta.D_{n5o}$	-	5.85	5.26
Dn50	m	0.24	0.31
Weight	kG	36.37	82.67

The result from Table shows that for the design situation the required weight of rock for toe protection is 36,5kG. This value is smaller than the applied value of rock for existing protection of toe (55kG). However at current situation the required rock size for structure is much more than that (82.7 kG). So the existing toe protection of revetments may not be stable under the design condition.

d. Some more required geometrical design dimensions for design of revetments.

d.1 Total thickness of the armour layers based on equilibrium of profile development.

Statically stable structures can be described by damage parameter S and dynamically stable ones by a profile.

Dynamically stable structures are those where profile development is accepted. Unit (rock, gravel and sand) are displaced by the wave actions until a equilibrium profile is reached. At this state the transport capacity along the profile ia reduced to a minimum.

The revetment usually has the initial profile of 1 on m (m=2, 3,4, 5...). After a sufficient number of waves (N=7500, recommended by Vander Meer) the profile is reached at equilibrium state, see Figure 5.13. The minimum thickness of the armour layers of revetment should be at least thick enough for ensuring that the under layer (normally filter layers) is not exposed. Based on the difference of initial and equilibrium profiles, the total required thickness of the armour layers can be determined.

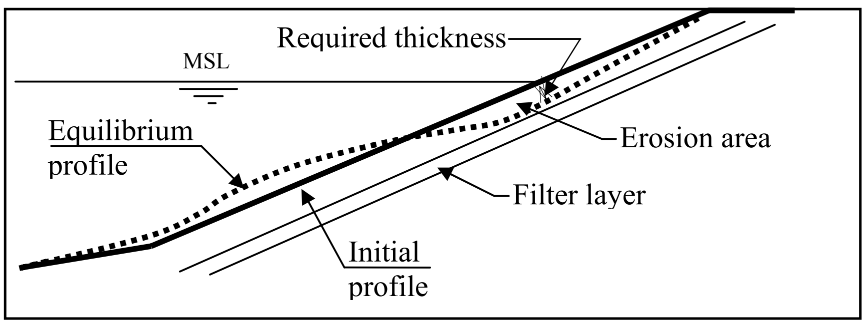
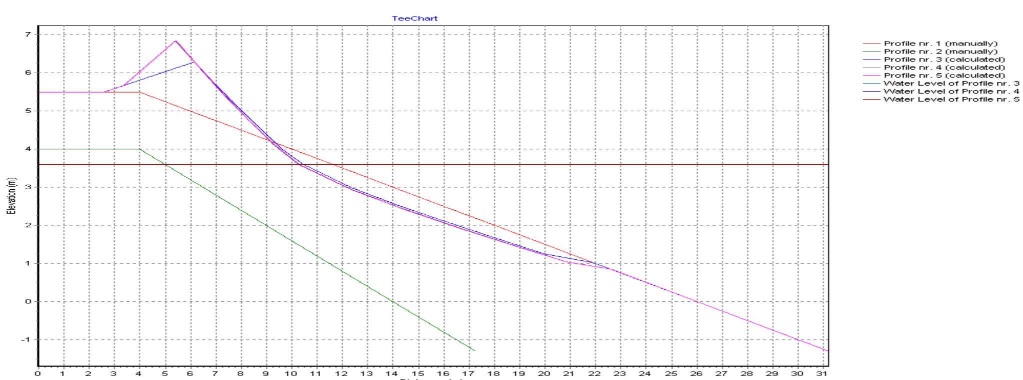


Figure 5.13: example of reshaped profile reached the equilibrium.

By using BREAKWAT model which based on the extensive models tests of Van der Meer (1998), the reshaped profiles can be simulated. The model simply gives the last state profile plot together with initial profile. For the given case of Namdinh revetment the reshaped profiles by using BREAKWAT are shown in Figure 5.14.

Based on the result the minimum thickness of armour layers should be 0.85 m in order to remain the safety for under layers of revetment and dike's body.



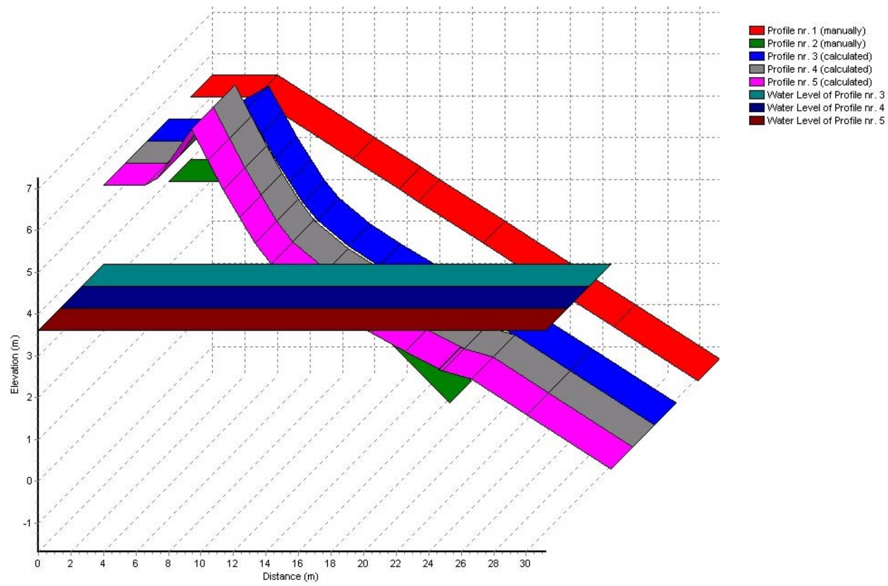


Figure 5.14: Simulation of Reshaped profiles by BREAKWAT

(based on Van der Meer 1998)

d.2 Required thickness of revetment by aspect of soil-mechanical stability.

When the water moves on a revetment structure it can affect the subsoil, especially, when this consists of sand. This effect is considered within the framework of the soil-mechanical aspects and can be of importance to the stability of the structure, see Figure 5.15. There are three aspects that will be discussed within the framework of soil-mechanical aspects: elastic storage; softening (liquefaction); and drop in the water level (see Pilarczyk et al.,1995). The background information can be found CUR169/RWS (2001).

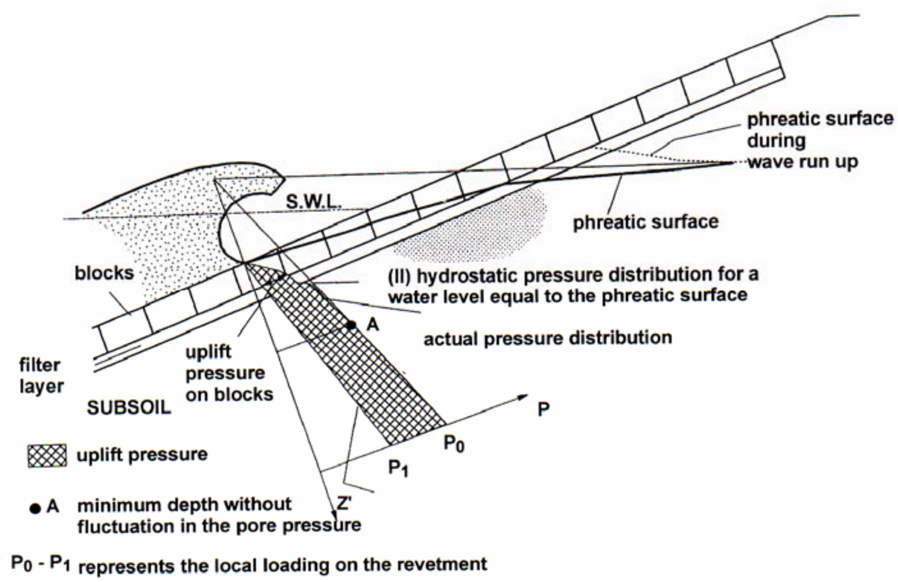


Figure 5.15: Pore pressure in the subsoil during wave run-down (Pilarczyk et al, 1998)

Elastic storage in the subsoil is connected with the permeability and stiffness of the grain skeleton and the compressibility of the pore water (the mixture of water and air in the pores of the grain skeleton). Because of these characteristics, wave pressures on the top layer are passed on delayed and damped to the subsoil of the revetment construction and to deeper layers (as seen perpendicular to the slope) of the subsoil. This phenomenon takes place over a larger distance or depth as the grain skeleton and the pore water are stiffer. Elastic storage can lead to the following damage mechanisms (Stoutjesdijk, 1996):

- lifting of the top layer;
- partial sliding of the top layer;
- sliding of the top layer;
- sliding of the subsoil (treated in geotechnical related stability section)

Checking these damage mechanisms will be done by using provided diagrams in Dikes and Revetments, 1995, which were basically developed for the concrete-filled systems. Applying for Namdinh revetment with rocks/bocks amour on the granualar filter layer, the minimum required thickness for a certain failure mechanisms will be a factor 1.1 to 1.2 lower due to the less integrity and stiffness of the system. And in case of systems placed on a filter layer as Namdinh revetments, the diagrams on the lifting and partial/total sliding of the top layer can be neglected. In that case the stability of the top layer must be treated in conjunction with the filter layer; the sliding of the subsoil will be the determinant factor. For the stability of the top layer, elastic storage is particularly of importance if the top layer is placed directly on the subsoil without granular filter.

Because the revetment construction consists of a top layer on a filter layer, the thickness of the filter layer may in these diagrams be partially or completely (depending on the type of revetment) added to the thickness of the top layer. The equivalent thickness is defined as: $D_{eq} = D + b/\Delta_t$, where D_{eq} is the equivalent thickness of the top layer, D is the real thickness of the top layer, b is the thickness of the filter layer and Δ_t, is the relative mass (weight) under water of the top layer. For given boundary condition of Namdinh revetment, the required equivalent thickness are in Table 5.19 :

Table 5.19: Required thickness of armour layer for Namdinh revetment to avoid geotechnical related failure of outer slope.

Failure mechanism	*equivalent thickness*	*b/Δ_t*	*Required thickness of armour layers*
lifting of the top layer	1.0	0.2	0.8
partial sliding of the top layer	1.0	0.2	0.8
sliding of the top layer;	0.7	0.2	0.5
sliding of the subsoil	1.0	0.2	0.8

Yields: the required thickness of armour layers is 0.80 m.

Liquefaction:

The difference between liquefaction and elastic storage is that with liquefaction, water overtension is connected with a plastic deformation of a grain skeleton instead of an elastic deformation. Water overtension through softening occurs when the grain skeleton deforms plastically to a denser packing. With regard to liquefaction, according to Dutch criteria, with a top layer on a granular filter there is generally no danger of liquefaction.

Drop in the water level: A drop in the water level may occur as a result of tide or a ship passing through a waterway or canal. As with placed stone revetments, the resulting uplift is especially dangerous when the top layer is sanded up due to which the permeability of the top layer may decrease in time.

There is no danger due to this phenomenon if $\frac{\Lambda \sin\alpha}{2} \leq \Delta D \cos\alpha$ (*)

in which: Λ is leakage length (m), α is slope angle ('), Δ is (representative) relative density of the top layer (-), D is (representative) thickness of the top layer (m).

The leakage length is determined as: $\Lambda = \sqrt{\frac{k_f D b_f}{k_{top}}}$

Where b_f is thickness of the filter layer (m); k_f is permeability of the filter; k_{top} is permeability of the top layer; D is thickness of the top layer.

For given case of Namdinh revetment, the leakage length Λ=0.1 m; then the condition (*) is satisfied.

d.3 Depth and width of toe protection

The protective required depth for the toe structure obviously should be at least equal to the maximum scour depth in front of the toe structure. The maximum scour depth is considered as an equilibrium scour depth during the life time of the structures. From practical experience it should lie from 0.5 to 1.0 times significant wave height (Pilarczyk et al, 1998). However the scour mechanisms and their depths depend on so many parameters and are very complicated. There are some studies on scour development around and near the toe of marine structures. These studies were based on both mathematical models and physical models. The scour mechanism around the toe of sloping structure is sketched in Figure 5.16. Unfortunately there are very few researches on that field for sloping structures like revetments. In this case there are no generally accepted methods for estimating maximum scour depth and other characteristics of the scour process (Sumer and Fredsoe, 2001).

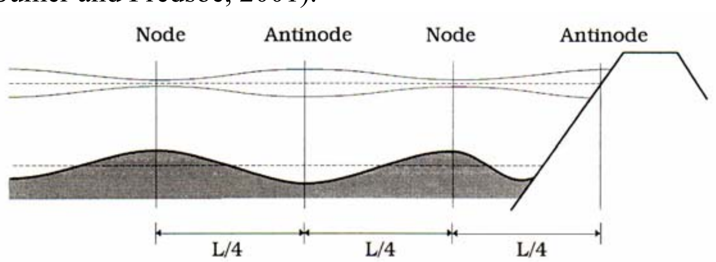


Figure 5.16: Scour mechanism near the toe of sloping structure.

According to the rules of thumb given by Coastal Engineering Manual (2001) regarding this issue are:

1. Maximum scour at the toe of a sloping structure is expected to be somewhat smaller than that calculated for a vertical wall at the same location and under the same wave conditions. Hence, a conservative scour estimate is provided by the vertical-wall scour prediction equations.

2. Structures with larger porosity will experience smaller wave-induced scour.

3. Scour depths are significantly increased when along-structure currents act concurrently with waves.

4. Obliquely incident waves may cause larger scour than normally incident waves because the short-crested waves increase in size along the structure. Also, oblique waves generate flows parallel to the structure.

+ Appling for the case of Namdinh revetment at the ***lowest water level*** as shown on Figure 5. 17, the maximum scour depth S can be determined by the following methods:

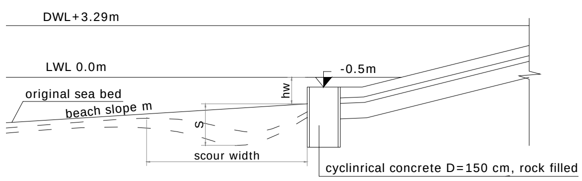


Figure 5.17: Schematization of scour mechanism at Namdinh revetment at LWL

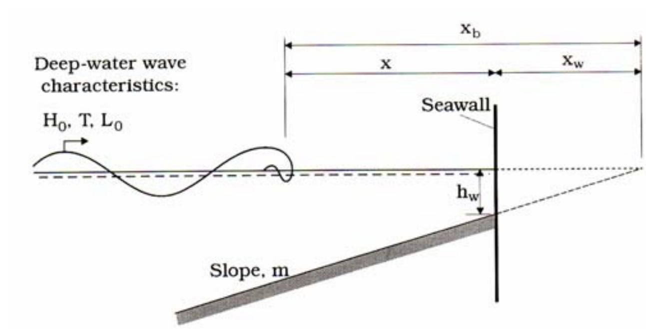
- ***McDonugal Method***

The scour depth in front of vertical seawall on the beach slope of m is:

$$\frac{S}{H_o} = 0.42m^{0.85}\left(\frac{Lo}{Ho}\right)^{0.2}\left(\frac{h_w}{H_o}\right)^{0.25}\left(\frac{H_o}{d}\right)^{1/3}$$

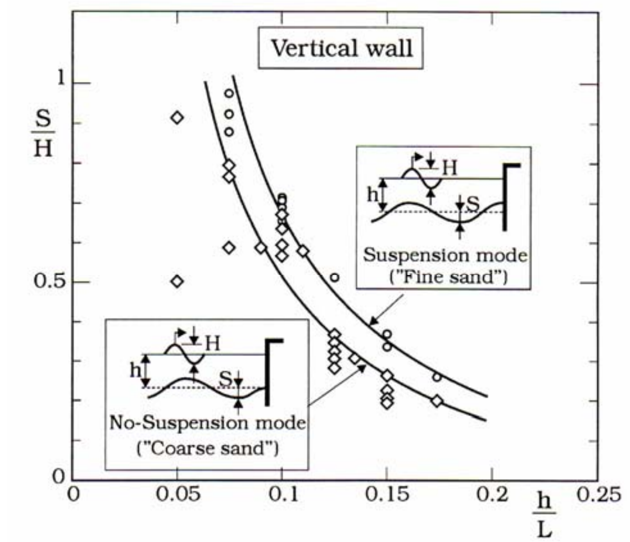
Where *S* is the scour depth at the seawall; *Ho, Lo* is wave characteristics in deep water, h_w is depth of the toe of structure and *d* is the sand grain size, see the schematized Figure 5.24. For given condition, S=1.2m. The detail is given by below Table.

Parameter	*Unit*	*Value*
Ho	m	4.6
Lo	m	164
hw	m	0.5
d	m	0.00025
m	%	0.01
S/H	-	0.3
S	m	1.2



- ***Method of Xie, 1981 (for vertical sea wall)***

According to Xie 1981, the maximum scour depth is given in the following Figure. In which H, L: local wave characteristics; h : local water depth; For given case of Namdinh revetment at Low water level, local wave length $L=T*(gh)^{1/2}=8.5*(9.81*1.6)^{1/2}=33.6$m; local wave height $H=0.55*h=0.55*1.60=0.9$m; h/L=0.05 => Yielding : ***S=0.85 m***.



- ***Method of Sumer and Fredsoe(2001) (developed for armour breakwater)***

This method given by series of graph on Figure 5.18. However it is applied for slope angle in the range of 30^o to 90^0. It suggest that when applied for the more gentle slope of 30^0, the curve of $\alpha=90^0$ should be applied scour depth then should be reduced by a certain appropriate factor.

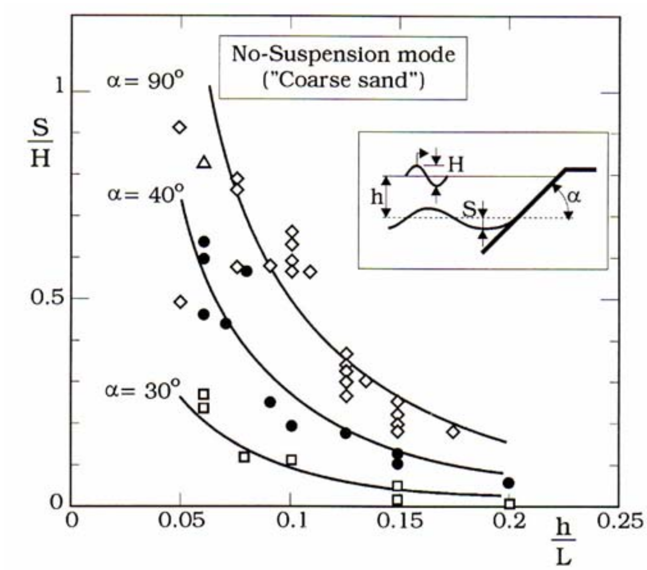
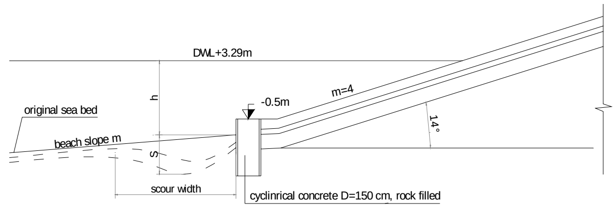


Figure 5.18: Maximum scour depth according to Sumer and Fredsoe 2001.

$$\frac{S}{H} = \frac{f(\alpha)}{\left[\sinh\left(\frac{2\pi h}{L}\right)\right]^{1.35}} \qquad \text{in which} \qquad f(\alpha) = 0.3 - 1.77e^{-\frac{\alpha}{15}}$$

Applied for Namdinh revetment:

+) At hign water level(DWL): the schematization of Namdinh revetment at DWL is shown attached Figure. Where $\alpha=14^o$ h=4.6m ; H=2.6m ; L=8.5*$(9.81*4.6)^{1/2}$=57m



Therefore:
S = 1.85m when applying for $\alpha=90^o$.
For $\alpha=14^o$, S=(14/90) * 1.85=0.30m

+) At low water level(LWL):

The schematization of Namdinh revetment at LWL refers to Figure 5.15. Where $\alpha=90^o$ (considered the cydinrical concrete as seawall); h=1.6m ; H=0.9m ; L=8.5*$(9.81*1.6)^{1/2}$=33m. Therefore: S=1.3m.

Summary:

- By applying various methods the different results of scour depth were given. The method of McDougal deals with deep water characteristic of waves while Xie's, Sumer&Fredsoe prefer local wave characteristic. In equation of McDougal, the grain size of sand near the toe was included. This may give more appreciate mechanism of scour.

- For initial design the maximum protective depth for toe structure should be selected by the largest value of maximum scour depths by applying these various methods then multiplies safety factor. For the case of Namdinh revetment, the initial depth for toe protection should be equal to: ***1.2 S_{max}=1.2*1.3=1.6m***.

- The development of scour mechanism is consecutively continuous process. Therefore after one year (or one storm season) the situation of the bed near the toe of the dike may differ from last year. As the consequence the boundary condition will differ from design situation also (normally getting more dangerous). In order to obtain the design situation annual observation should be made. Based on the observation the maintenance can be provided in proper way annually.

Width of toe protection

It is imperative to construct a protective for toe protection. This protective layer may be constructed in the form of a berm, or an apron (Figure 5.16). The protection of the berm must be designed in such a way that it will remain intact under the wave and currents forces. It should also be flexible enough to form to an initially seabed. With this counter measure, scour hole can be minimized, however, not entirely avoided. Some scour may occur around the edge of the protective layer, and consequently, armour stones will slump down into the scour hole. This later process will lead to the formation of protective slope as a desirable effect for fixing the scour.

One of the most important design concerns is the determination of the width of the protection of the toe. This width should sufficient large to ensure that some portion of the apron remain intact, providing adequate protection for the stability of the breakwater.

According to Mumer and Fredsoe(2000), based their measurements of the width of the scour hole at the breakwater, recommend that the width of scour protection(width of apron) should be equal to the width of the scour hole. The width of scour hole is as follows:

Vertical wall : W= 1.0x(L/4)
Sloping wall : W= 0.6(L/4) for slope of 1: 1.2
W= 0.3(L/4) for slope of 1: 1.75

For the case of Namdinh revetment with slope of 1:4, much milder than the given cases. Therefore the scour protection should be less than that by a reduced factor. The reduced factor is chosen as proportion of slope angle between sloping case and vertical case (can be interpolated from these above given cases). Hence, the width of toe protection should be: $L_{prot.}$= W= 1.0x(L/4)x($14^0/90^0$)=0.16 x (L/4). Then:

L_{prot} =W= 0.16 x (33/4) = 1.32 m for the case of Low water level; and

L_{prot} =W= 0.16 x (57/4) = 2.28 m for the case of High water level
The selected width of toe protection L_{prot}=2.5m

e. General conclusion

- In the present designs of sea dikes Russian formulae as well as the Hudson formula are used. All these formulae are originally developed for riprap and/or rubble mound structures, which are based on the weight of elements. However, these formulae have also been applied in the present designs for calculation of thickness of pitched stone and block revetments. No comparison between the Russian and Hudson formula are made. There also no official recommendations are formulated on the choice of one of these formulae for the design of revetments. The choice of the formula is left up to the designers. However, the designers have no sufficient information on the backgrounds of these formulae, and they are making a choice based on their own individual preference. Due to this fact, each design office use different formulae and provides different stability calculations and, as a result, the comparison of various designs is not possible.
- The valid ranges of using given formulae should be included in design guide line.
- The design of block revetments should be based on the thickness of a block and not on the weight of a block. That provides possibility to produce a concrete block of a high stability but with an acceptable (lower) weight for a handling by individual persons.
- When using interlock blocks the stability may increase by a factor 1.5 to 2 in comparison with loose blocks. In such a case, special attention should be paid to a stable design of revetment foundation (sub-layers and subsoil). Even small settlement will lead to creating of cavities under the interlocked blocks. These cavities will be not visible (not inspectable) until damage take place.
- In order to be more stable when using placed concrete blocks the revetment can be strengthened by washing in the interspaces by a coarse sand (improve the interfaced friction between block).
- Design guidelines should be improved and formulated including the selection of proper design formulae for various revetment types, including stability of sub-layers and

subsoil, and also including design recommendations concerning toe protection and transitions, selection the proper dimensions for the structures.

f. Some Dutch types of toe protection

The composition and dimension of toe protection can be referred to some Dutch types as shown in figure 5.19. More detail should be looked at Dikes& Revetments of Pilarczyk et al, 1998.

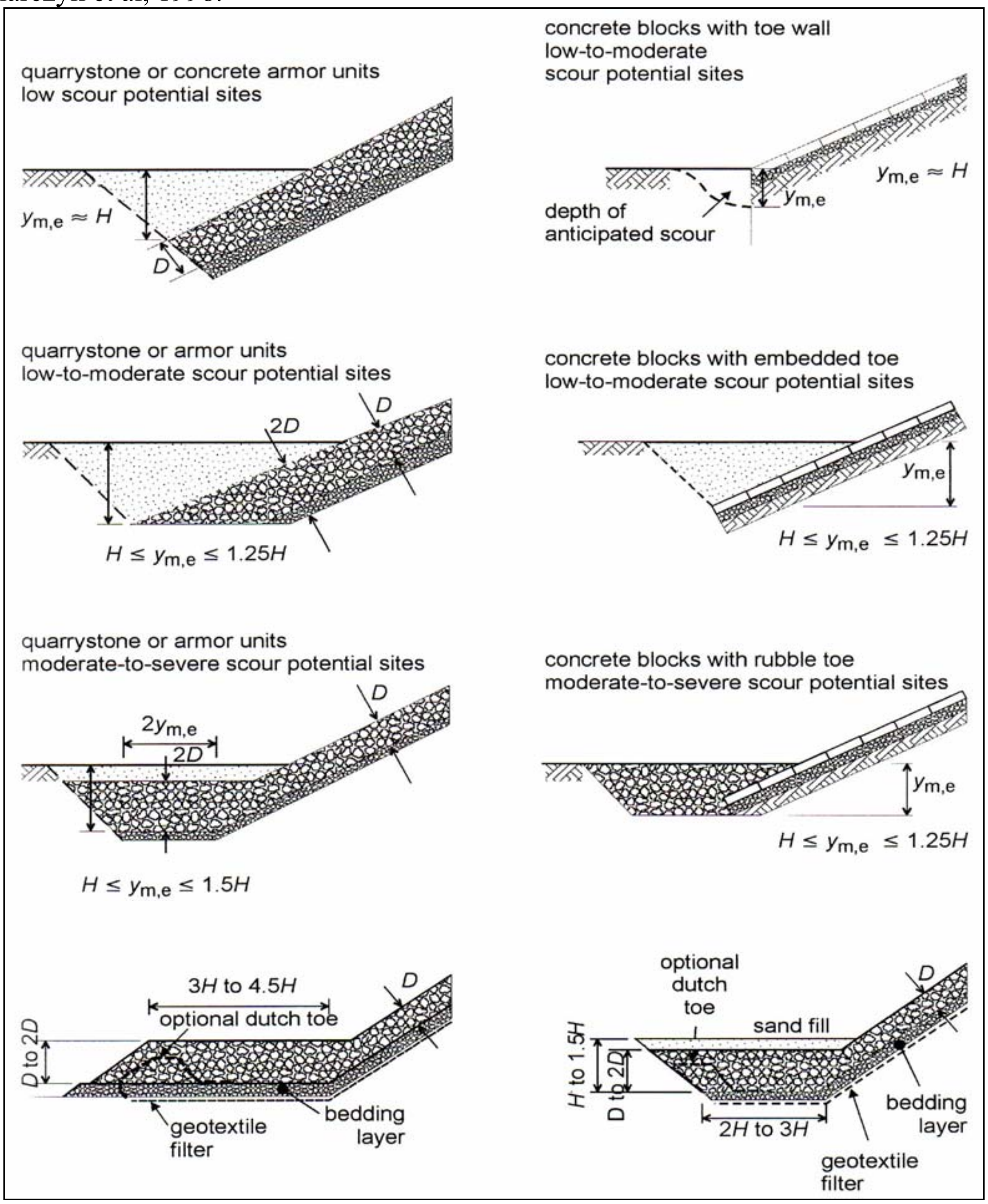


Figure 5.19: Some alternative toe protections (Pilarczyk et al, Dikes& Revetments, 1998)

($y_{m,e}$=scour depth; H= local wave height).

5.2.3 Geotechnical related stability of the dikes

5.2.3.1. Generally geotechnical conditions, limit states and boundary conditions.

The geotechnical conditions are based on soil investigations at the site, laboratory testing and geotechnical calculations of the samples. The outcomes for geotechnical conditions are the type of soil and its distribution along coastline and its varying by depth, the mechanical properties of various soils with respect to their strength and deformation characteristics…

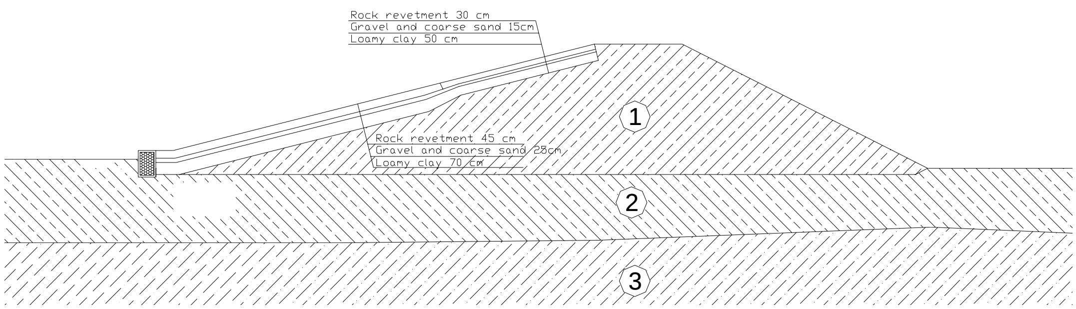


Figure 5.20: geotechnical geometry of Namdinh dike section.

For the representative cross section of Namdinh sea dikes, the geotechnical conditions for safety calculations are based on the data of soil investigation and design properties of materials during final design process, given in technical design document, 1996. See Figure 5.20 and Table 5.20.

Table 5.20. Material properties of dike's body and subsoil at HaiTrieu section

Parameter	Not.	Unit	(1)	(2)	(3)	(4)	(5)	(6)
Soil unit weight (unsaturated)	$\gamma_{unsat.}$	kN/m^3	15	16.5	19	17	21	23
Soil unit weight (saturated)	$\gamma_{sat.}$	kN/m^3	18	20	21	19	24	25
Permeability	k	m/s	4.10^{-7}	3.10^{-6}	10^{-5}	5.10^{-8}	5.10^{-2}	1
Young's modulus	E	kN/m^2	15.10^3	10^4	25.10^3	15.10^3	10^6	2.10^6
Possiom's ratio	μ	-	0.30	0.31	0.3	0.32	0.15	0.1
Cohesion	C	kN/m^2	19	7	1	16	0	0
Friction angle	φ	Degree	22.5	11.5	30	20	32	35

Layer (1) : Dike's body – loamy sandy clay
Layer (2) : Upper subsoil - sandy clay
Layer (3) : Under subsoil – fine sand
Layer (4) : Under layer of outer slope – loamy clay
Layer (5) : filter layer – sand + gravel
Layer (6) : Armour layer – random filled stone

Limit states and boundary condition

Sea dike are the coastal structures which have to withstand a combination of actions induced by waves, currents, differences in water levels, seismiticy and other specific loadings (Such as ship collision or surcharges). These actions, including the dead-weight of the structure, have to be transferred to the subsoil in such a way that:

1. The deformations of tile structure are acceptable and

2. The probability of instability is sufficiently low.

The transfer of actions through the structure to the subsoil involves changes in soil stresses (pore water pressures and effective stresses) in the soil layers. In case of dikes, the sloping structures, the soil stresses are also in the structure itself. Particularly in soft soils the stress changes will gradually develop during a long period of time. Due to these changes in soil stresses the underlying and adjacent soil layers will deform vertically and horizontally while the shear strength of the soil may be reduced. As a consequence dike and its component structures built on top of the soil layers will deform too or may even lose its stability. This applies to both the design conditions under extreme loadings and constructed condition with the loadings during the construction period of the structures.

The changes in soil stresses and the associated deformations not only depend on the hydraulic loadings but also on the geometry of the dike (e.g. slope steepness, width of the crest), the structure weight and on the permeability, stiffness and shear strength of the subsequent structure and soil layers as well. For this reason the design and safety assessment of sea dikes has to be based on an integral approach of the interaction between the structure and the subsoil. Traditionally, the main geotechnical limit states are evaluated for the dikes are:

- Macro-in stability of slopes due to failure along circular or straight sliding surfaces;
- Settlements (and horizontal deformations) due to the self weight of the structure;
- Micro-instability of slopes caused by seeping out of groundwater;
- Piping or internal erosion due to seepage flow underneath the structure;
- Liquefaction caused by erosion (flow slides) or by cyclic loading (wave actions or earthquakes);
- Erosion of revetments at the outer slopes (or under water slopes) due to instable filters or local failure of top layer elements.

The criteria for most of the above geotechnical related failure modes which apply by both Vietnamese and Dutch design codes are not different under deterministic approach. For this reason the comparison between applying the two design codes are not mentioned in this subsection. However the safety assessment of the above failure mechanisms are investigated in order to concludes how safe the dikes in Namdinh are.

Loading boundary is applied for extreme condition. The sea water level is at +3.30 meter (+MSL) as the combination of high tides and design storm. The applied design wave height is 2.6m with design wave period of 10 second. Exceptionally, for checking stability of outer slopes the considerations are made with quick draw down of sea water level from DWL to the lowest level at +0.5 m (+MSL); and assuming that the sea state is quiet which provides no waves attack. The water level at leeside of the dikes is considered as the possible lowest value at the level of inland surface. In addition on top of the dike the normal traffic loading and stored stones are taken in to account.

Geometry of the dike section for calculation is applied at present situation containing dike's body with slope protections and subsoil. The subsoil includes 2 layers. Left and right boundaries of geometry are selected at 15 meters, which is equal to 2 times of dike's height, from interacted point of dikes slope and first subsoil surface at each side. This is based on the advice of selection of effective geometrical boundaries, see Vietnam design standard for sea dikes.

The more details of the boundary conditions is described in Figure 5.21.

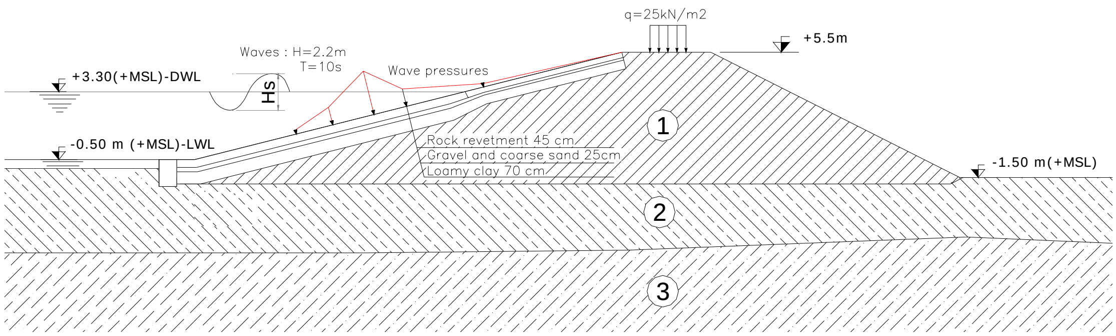


Figure 5.21: Boundary condition for calculations of geotechnical related problems.

In this section the safety analysis for most of the above geotechnical related failure mechanisms are carried out by applying numerical method for solving geotechnical problems. Exceptionally for piping safety related, the analytical method is used, based on the actual formulae for piping under the dikes.

Two advanced geotechnical packages PLAXIS(1)(see Appendix 1) and GEO-Slopes(2) (see Appendix 2) are used as the calculation tools in order to obtain the solutions. The related equations and detailed results for the problems are presented in appendixes. The more details about geotechnical modeling with PLAXIS and GEO-Slopes see also R.B.J. Bringkgreve, PLAXIS-Finite element code for soil and rock analyses, 2D-version 8, 2001 and GEO-Slope user's guide, GEO-slope international Ltd., 2001.

In this chapter summarized results of considered problems will be presented. Based on these results some conclusions for related safety of the dikes will be given out.

5.2.3.2 Analyses of seepage through the dikes and subsoil.

Seepage via the dikes and subsoil are obtained by using the calculation procedures of steady-state groundwater flow calculation module with PLAXIS. The flow in a porous medium of the soil was described by Darcy's law and formulated by finite element method.

For given input boundary conditions, the results show that the seepage flows out at lower part of inner slope of the dike from level +0.5 (+MSL) to below, see Figure 5.22. In zone A, lower part of inner slope combining with subsoil surface, there is high concentration of outflow with maximum velocity of 2×10^{-6}m/s, see Figure 5.23. Nearby the slope surface, the hydraulic gradient is at high value of 0.5 (it equals to steepness of inner slope). On the other hand, the phreatic line inside dike's body is still relative high despite providing the clay layer at outer slope side.

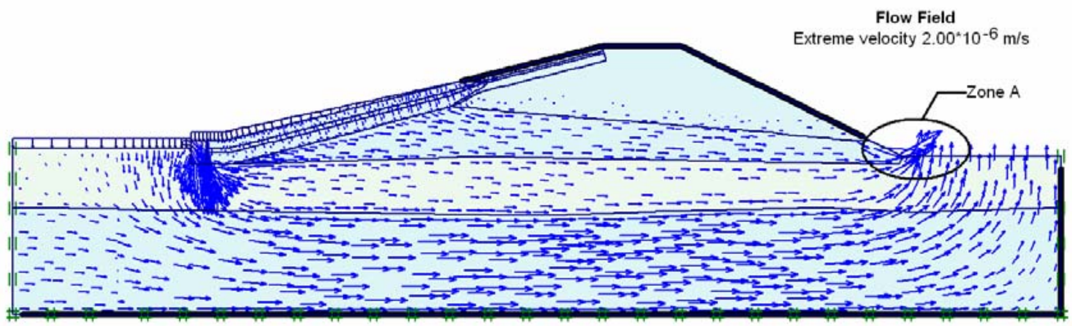


Figure 5.22. Seepage flow field

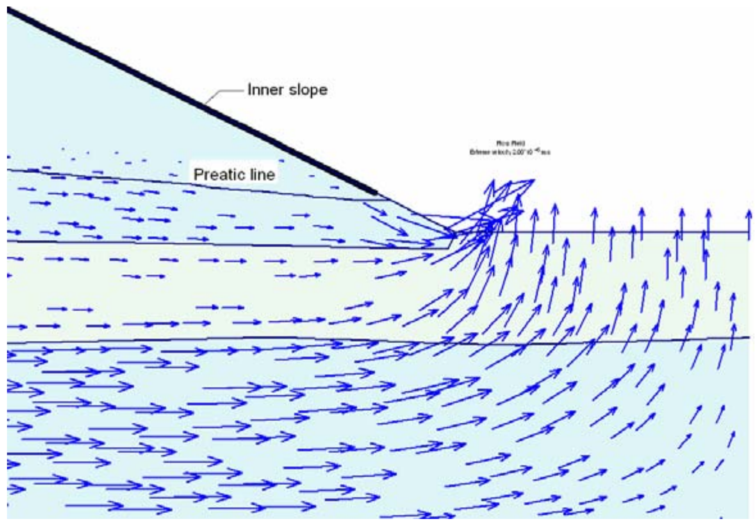


Figure 5.23. Flow field of seepage in zone A

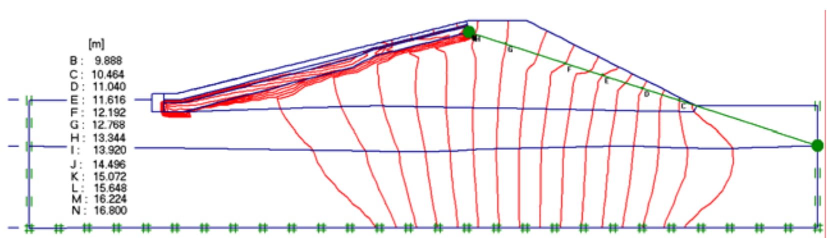


Figure 5.24. Active groundwater pressures

Based on the results of seepage analysis we can conclude that :

- The thickness of clay layer at outer slope (layer No 4) is not sufficient enough to lower the phreatic line inside dike's body. The relative high phreatic level inside the dike causes the more saturated area of dike's body. This leads to the higher value of active pore pressure and the smaller effective stress in the soil. As the consequence the stability of the dike may be getting less at design boundary condition.
- The permeability of dike's material is not low enough to avoid the outflow at inner slope while there is not any filter layer and/or drainage tool of inner slope.

This may lead to loss of soil particles and micro-instability of the area due to high hydraulic gradient.

In order to avoid the instability of the dike due to seepage, for the existing dike the filter layer and drainage tools should be added at lower part of inner slope from level +0.5 to below. For new dike design, the consideration of seepage analysis should be made when selecting the body soil and geometrical composite dimensions of dike's cross section.

5.2.3.3 Analyses of stress-strain and displacements.

Analysis of stress-strain and displacements were based on the basic equations of static deformation of the soil body, which formulated within the framework of continuum mechanics with small strain, and the well-known Biot's consolidation theory combining with Terzaghi's principle of the relation betweem total stress, effective stress and pore pressure. By applying these above theory, Plaxis solved the geotechnical problems by finite element formulation with the assumtion of small strain and discretised the continuum description, see also R.B.J. Bringkgreve, PLAXIS-Finite element code for soil and rock analyses, 2D-version 8, 2001. The short over view of the basic equation and finite element formulation will be presented in Appendix 1.

In this section, the results of stresses, strains, deformations for given boundary condition of Namdinh dike are summarised. The problems are solved by using PLAXIS.

Considered loads:

- Principal loading parameters : Weight of soil and material, sea water pressures, water wave pressures, traffic load,
- Secondary loading parameters: Pore pressures, initially steped deformation.

Influent characteristics : Soil compressibility and permeability, thickness of pressible layers

Dike response: Lowering of the dike crest (totally vertical deformations) and horizontal deformations.

Outcome: the results of calculation are all kind of main geotechnical related components for whole calculated area, displayed on a plot of geometry. They present under the clearly numerical graphic solution with 3 optional result modes: contour lines, mean shadings or vectors (arrows).

The positive sign notes for the displayed result components, exceptional for any total components, are: X-axis: Positive direction is from left to right; Y-Axis: Positive direction is upward. For the total components, the results show absolute value.

In Figure 5.25 shows results of Total displacements of the problem in 3 result modes as a representative.

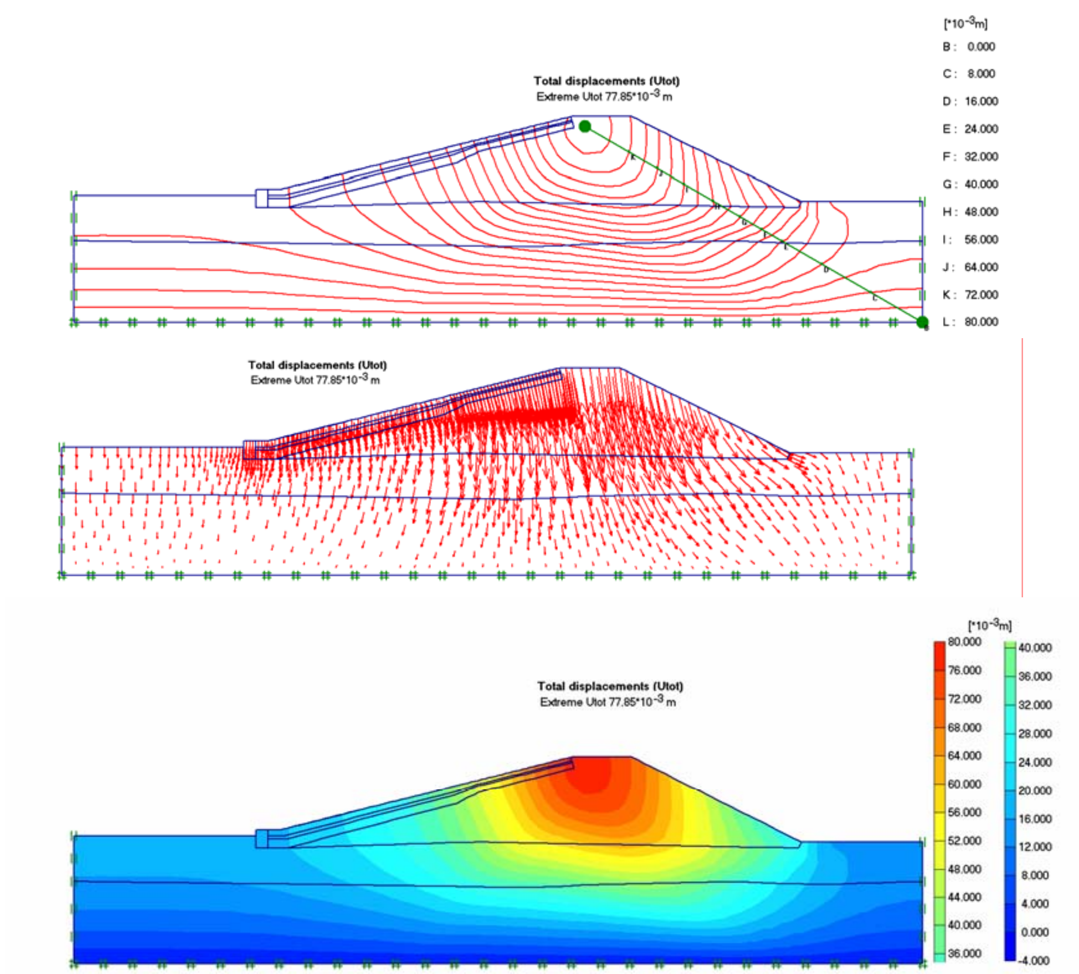


Figure 5.25: Total displacements of the problem in 3 result modes

For all the considered results of the problem will be presented in Appendix 1. Including:

- *Deformation mesh*, which shows finite element mesh in the deformed shape, the extreme value of deformation is 7.8 cm at the crest of the dikes.

- *Displacements U*, which gives the accumulated displacements compiled from horizontal, vertical displacements *(Ux, Uy)* at all nodes at the end of calculated processes. The extreme value of displacement is :

Extreme total Dis.: *U* = *77.85*x10^{-3}m at crest of the dike

Extreme horizontal Dis.: Ux=29.35x10^{-3}m at lower part of inner, near interface of dike's body and subsoil.

Extreme vertical Dis.: Uy=-76.78x10^{-3}m at crest of the dike.

- *Total strains* ε, the accumulated strains in the geometry at stress points at the end of the calculation step, total strains are represented as Principal directions (principal strains), Volumetric strains (ε_V,) or equivalent Shear strains (ε_S) by selecting the appropriate displayed options. Note that compression is considered to be negative.

There are some small areas, which provided positive strains in order of 0.2 to 0.5 %, located at outer slope around DWL and at lower end of inner slope. This means that the tensile stresses are presented there.

- *Effective stresses,* being represented as Principal directions (principal stresses) which represent the average of the principal effective stress in the element, Mean stresses or Relative shear stresses which gives an indication of the proximity of the stress point to the failure envelope. The relative shear stress is defined as:

$\tau_{ref} = \dfrac{\tau}{\tau_{\max}}$ where τ is the maximum value of shear stress (equals to the radius of the Mohr stress circle). The parameter τ_{max} is the maximum value of shear stress for the case where the Mohr's circle is expanded to touch the Coulomb failure envelope keeping the intermediate principal stress constant.

The extreme effective stress is -178.83 kN/m^2 inside dike's body. Extreme relative shear stress is 1.0 along the outer slope, under armour layer of revetment.

- *Total stresses (effective stresses + active pore pressures):* Total stresses can be represented as Principal directions (principal stresses), Mean stresses or Deviatoric stresses. Note that pressure is considered to be negative when calculating stresses and compression stress is negative. *Extreme value of total stresses:* -313.72 kN/m^2

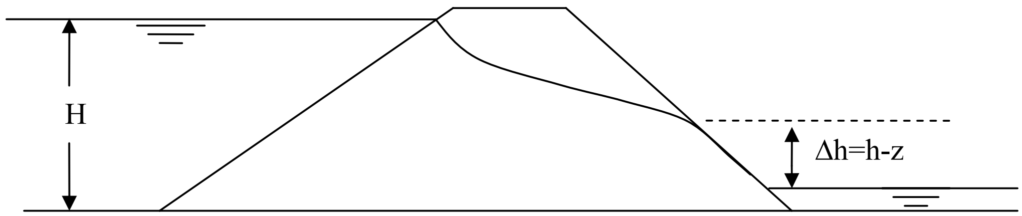
- *Plastic points*, these are the stress points in a plastic state, displayed in a plot of the un-deformed geometry. The plastic stress points are indicated by small symbols that can have different shapes and colours, depending on the type of plasticity that has occurred. A red open square indicates that the stresses lie on the surface of the Coulomb failure envelope. A white solid square indicates that the tension cut-off criterion was applied.

The Coulomb plastic points are particularly useful to check whether the size of the mesh is sufficient. If the zone of Coulomb plasticity reaches a mesh boundary (excluding the centre-line in a symmetric model) then this suggests that the size of the mesh may be too small. In this case the calculation should be repeated with a larger model.

5.2.3.**4. Micro instability**

Micro-instability will be checked for representative section of Namdinh sea dikes. The analysis is based on the results of stress- strain and groundwater flow calculations.

a. Micro-instability of inner slope due to seepage



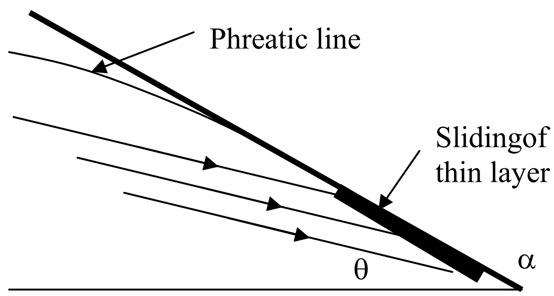


Figure 5.26 Micro-instability at inner slope.

The situation is sketched in Figure 5.26. For Namdinh dikes, the inner slope is above water level. The groundwater flows out at inner slope, see also section 5.2.3.2 and Figure 5.23. According to TAW 2000 the criteria for stability without safety factor can be applied by:

$$\frac{2cd}{\gamma_{m,c}}+\frac{\rho_g g}{\gamma_{m,\rho}}\Delta x d\cos\alpha+\frac{\rho_g g}{\gamma_{m,\rho}}\Delta x d\sin\alpha\frac{\tan\phi}{\gamma_{m,\phi}}\geq\gamma_n\gamma_d(\Delta h-\frac{1}{2}\Delta x\sin\alpha)\frac{\rho_w g}{\gamma_{m,\rho}}\Delta x$$

In which:

h is ground water level in relation to reference level (m), this can be determined from calculation model of groundwater flow.

z is level of inland water level in relation to reference level (m).

d is assumed layer thickness, perpendicular to the slope (m)

c is cohesion for the soil [kN/m^2 per m^1]

g is force of gravity [m/s^2]

$\gamma_{m,c}$ is (= 1.250 material factor for the cohesion [-]

$\gamma_{m,\phi}$ is (= 1.1) material factor for the angle of internal friction [-]

ϕ is angle of internal friction of the clay [^{0}]

This equation can be solved if Δh = (h-z) is known. The term Δx stands for the width of a slice of ground, measured parallel to the slope and beginning at the toe of the slope. For the total safety ($\gamma_d \gamma_\eta$) with perpendicular stability 2.0 is assumed, see TAW, Safety Monitoring Guidelines

c = 19 kN/m^2 per m; tan ϕ =0.22 (ϕ=12.5^0)
d = 04; 0.8; 1.2; 1.6; 2.0 m
g = 9.81 m/s^2
ρ_g = 1630 kg/m^3 ; ρ_w = 1000 kg/m^3
slope = 1:2

Applying the equation for the given case, the admissible head for avoiding instability as shown in Figure 5.27.

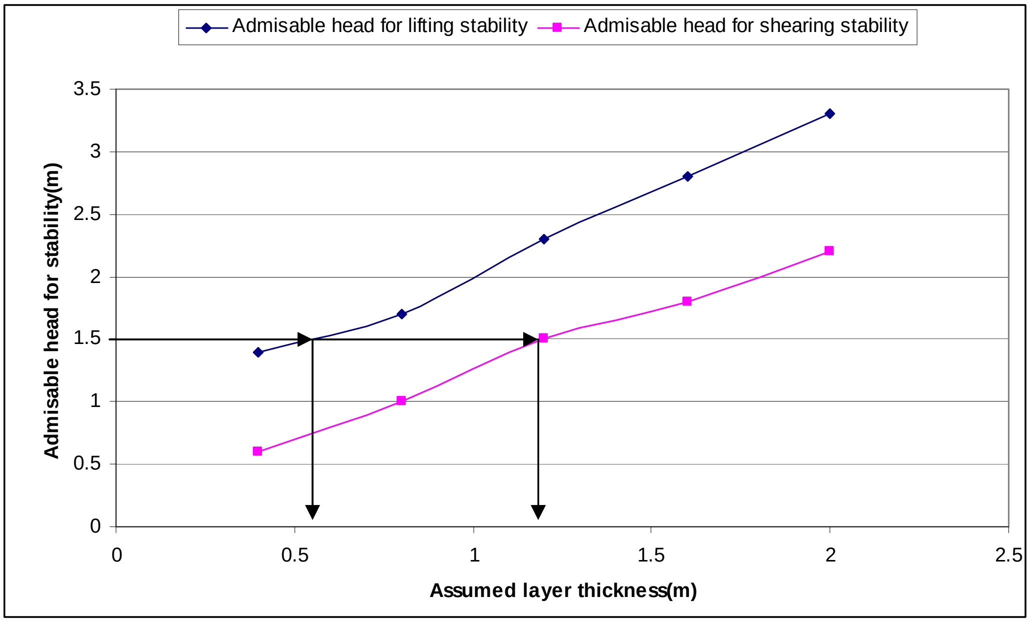


Figure 5.27: Admissible head for avoiding instability

For given case, Δh=1.5m, based on the graph in Figure 5.27, the inner slope could be unstable due to lifting with the layer thickness of smaller than or equal 0.55m, and due to shearing with the layer thickness of smaller than or equal to 1.2m.

b. Local instability due to exceeding of critical stress-strain states (bearing capacity)

For some small areas inside dike's body which present the stress states exceeding the bearing capacity of the soil are considered as unstable areas. As mentioning in previous section, the relative shear stress is defined as the ratio of the τ, the maximum value of shear stress (equals to the radius of the Mohr stress circle) which may occurs, and τ_{max}, is the maximum value of shear stress for the case where the Mohr's circle is expanded to touch the Coulomb failure envelope. Therefore the stress points which have relative shear stresses larger than 1 should be considered as unstable points.

For given loading boundary conditions of Namdinh dikes, due to the actions of the external loads and self weights the plastic points are developed and presented as the red square points in Figure 5.28. These points are considered that working state of material passing elastic state and reaching to plastic state. The development of these areas is considered as the occurring of locally failure mechanism. In this case the total area covering the plastic points is considerably so large, about 15% area of the cross section roughly, that to threat stability of the dike's body under design conditions.

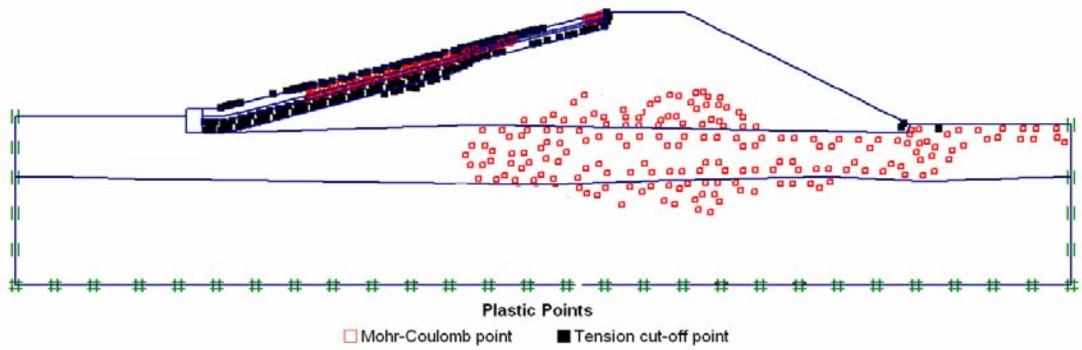


Figure 5.28: Plastic and tension cut-off point develop in dike body and subsoil.

c. Local instability due to occurring of tensile stress

In some practical problems, an area with tensile stresses may develop. According to the Coulomb envelope shown in Figure 5.29 this is allowed when the shear stress (given by the radius of Mohr circle) is sufficiently small. However, the soil surface near a interface in clay sometimes shows tensile cracks. This indicates that soil may also fail in tension instead of in shear. This behaviour can be analyzed by selecting the tension cut-off option in PLAXIS. In this case Mohr circles with positive principal stresses are not allowed. When selecting the tension cut-off the allowable tensile strength may be entered. For the Mohr-Coulomb model the tension cut-off is selected with a tensile strength of zero.

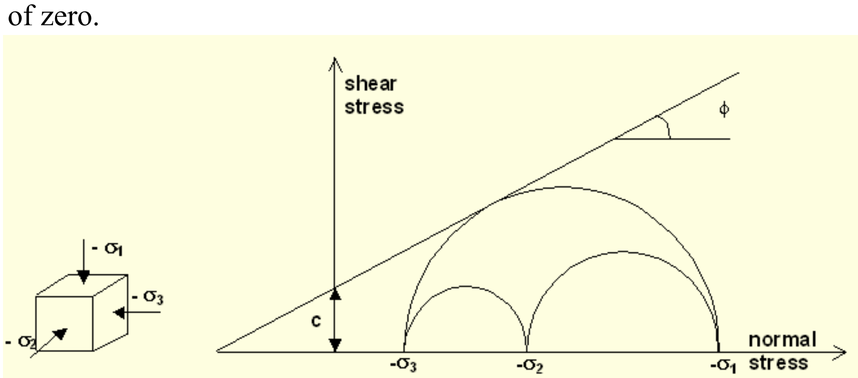


Figure 5.29: Stress circle touches Coulomb's envelope.

For the given boundary condition of Namdinh dikes, the tension cut-off points appear as black square points shown in Figure 5.29. It is clear that the tension points develop mostly in the narrow zone nearby outer slope boundary. The most dangerous case is the occurring of tensile stresses in clay layer under the revetment and in the thin zone of dike's body along outer slope. These occurring may cause the cracks of the clay layer and loosening of soil in the tensile zones.

d. Settlements and deformations

The maximum deformation and settlement occur at dike's crest. According to the deformed plot, the extreme value of vertical displacement is around 8 cm while the height of the dikes is 6.5 meters. Therefore the relative vertical deformation is 1.23%. This is accepted by Vietnam Design standard for Dams and Soil structures (allowed value is 1.5%). However this value of settlement must be taken in to consideration in design phases when selecting the design crest level of the dikes.

5.2.3.5 Overall safety analysis

It is necessary to know the global safety of the whole soil body, includes the dike body and subsoil, at working state under design condition. The overall safety analysis archiving with global safety factor will express this mean.

In structural engineering, the safety factor is usually defined as the ratio of the collapse load to the working load. For soil structures, however, this definition is not always useful. For the dikes, for example, most of the loading is caused by soil weight and an increase in soil weight would not necessarily lead to collapse. A more appropriate definition of the factor of safety is therefore:

$$\text{Safety factor(SF)} = S_{\text{maximum available}}/S_{\text{needed for equilibrimn}}$$

Where S represents the shear strength. The ratio of the true strength to the computed minimum strength required for equilibrium is the safety factor that is conventionally used in soil mechanics. By introducing the standard coulomb condition, the safety factor is obtained:

$$SF = \frac{c_i - \sigma_n \tan\varphi_i}{c_r - \sigma_n \tan\varphi_r}$$

Where c_i and φ_i are the input strength parameters and σ_n is the actual normal stress component. The parameters c_r and φ_r are reduced strength parameters that are just large enough to maintain equilibrium. The principle described above is the basis of the method of Phi-c-reduction that can be used in PLAXIS to calculate a global safety factor. In this approach the cohesion and the tangent of the friction angle are reduced in the same proportion:

$$\frac{\tan\varphi_{input}}{\tan\varphi_{reduced}} = \frac{c_{input}}{c_{reduced}} = \sum Msf$$

The reduction of strength parameters is controlled by the total multiplier $\sum Msf$. This parameter is increased in a step-by-step procedure and, as the consequence; the strength parameters $\tan\varphi$ and c of the soil are successively reduced until failure of the soil body occurs. The safety factor is then defined as the value of $\sum Msf$ at failure, provided that at failure a more or less constant value is obtained for a number of successive load steps.

$\sum Msf$ is set to 1.0 at the start of a calculation to set all material strengths to their original values. The load advancement number of steps procedure is used in order to perform a Phi-c reduction calculation .The incremental multiplier *Msf* is used to specify the increment of the strength reduction of the first calculation step. The strength parameters are successively reduced automatically until the final step has reaches and results in a fully developed failure mechanism. Then factor of safety is given by:

$$SF = \frac{Strength_{available}}{Strength_{at\_failure}} = value\_of\_\sum Msf\_at\_failure$$

The safety factor calculation by using PLAXIS with Phi-c reduction procedure can be seen more detail in PLAXIS Manual. The *Phi-c reduction* approach resembles the method of calculating safety factors as conventionally adopted in slip-circle analyses. For a detailed description of the method of Phi-c reduction see Brinkgreve et al, 1991, Non-linear finite element analysis of safety factors.

Applying for the case of Namdinh dikes with given boundary conditions the results shows in Figure 5.30 and 5.31.

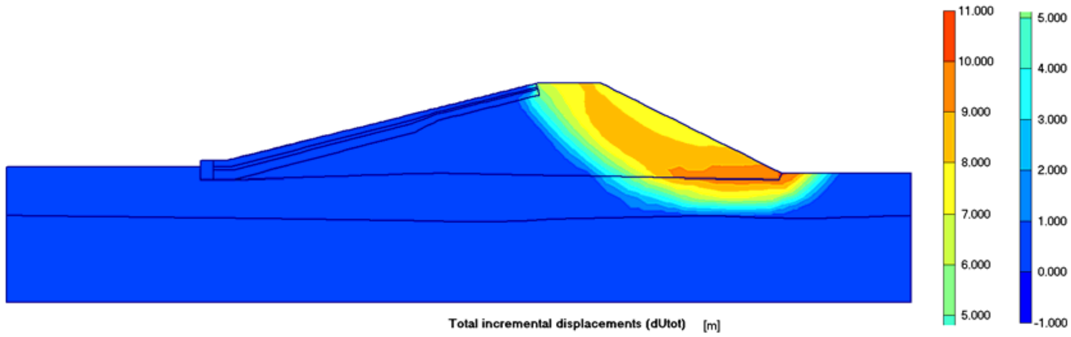


Figure 5.30: Total incremental displacements indicating the possibly applicable failure mechanism of Namdinh dike.

In Figure 5.30, the total incremental displacement at final step is presented. These do not have a relevant physical meaning, however, give the indication and impression of the likely failure mechanism.

The safety factor can be obtained from calculation by represented final value of ∑Msf, provided that this value is indeed more or less constant during the previous some steps (see Figure 5.31). The Figure shows the safety factor at every the loading step in relation of the total displacements. However the displacement is not relevant, it indicate whether or not the failure mechanism has developed.

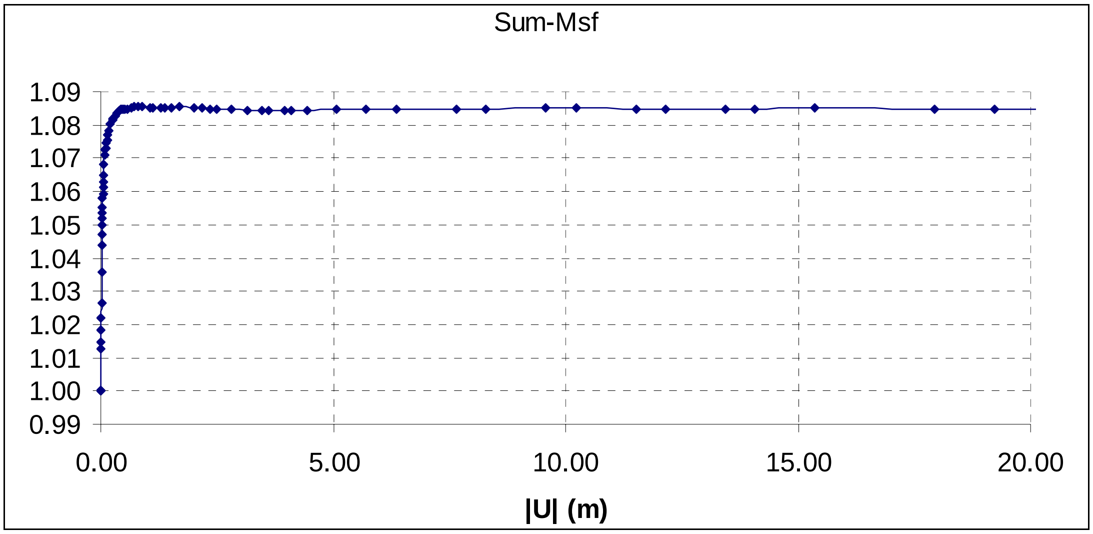


Figure 5.31: Safety factor in relation of loading steps and displacement as well

In this case, the global safety factor of the Namdinh dike is around ***1.085***. This means that the working state of the dike under design condition seems to reach limit state and the dike is in low reliability of stability.

5.2.3.6 Slope stability analysis

In additional to global safety analysis the estimation of slope stability is also important for soil sloping structures. For sea dikes the considerable failure mechanisms are circular shear failures of outer and inner slopes (slip circle failures). It happens if the actual shear stress along a potential failure planes exceed the shearing resistance along that plane.

For the given boundary conditions of a representative cross section of Namdinh dikes, the assessment of slope stability will be determined by applying Generalized Limit Equilibrium (GLE) method and Bishop's method. The calculation procedures are done by using GEO-slope model with Slope/w module which will be presented briefly in Appendix 2.

Summarized results

For outer slope, the calculation was made for the two cases of sea water condition. Case 1 with sea water level is at DWL (+3.3m) and case 2 is with quick draw down of sea water level for DWL (+3.3m) to LWL (+0.5m). The result is shows in Figure 5.32. The safety factor of outer slope is **1.898** for case 1; and **1.289** for case 2.

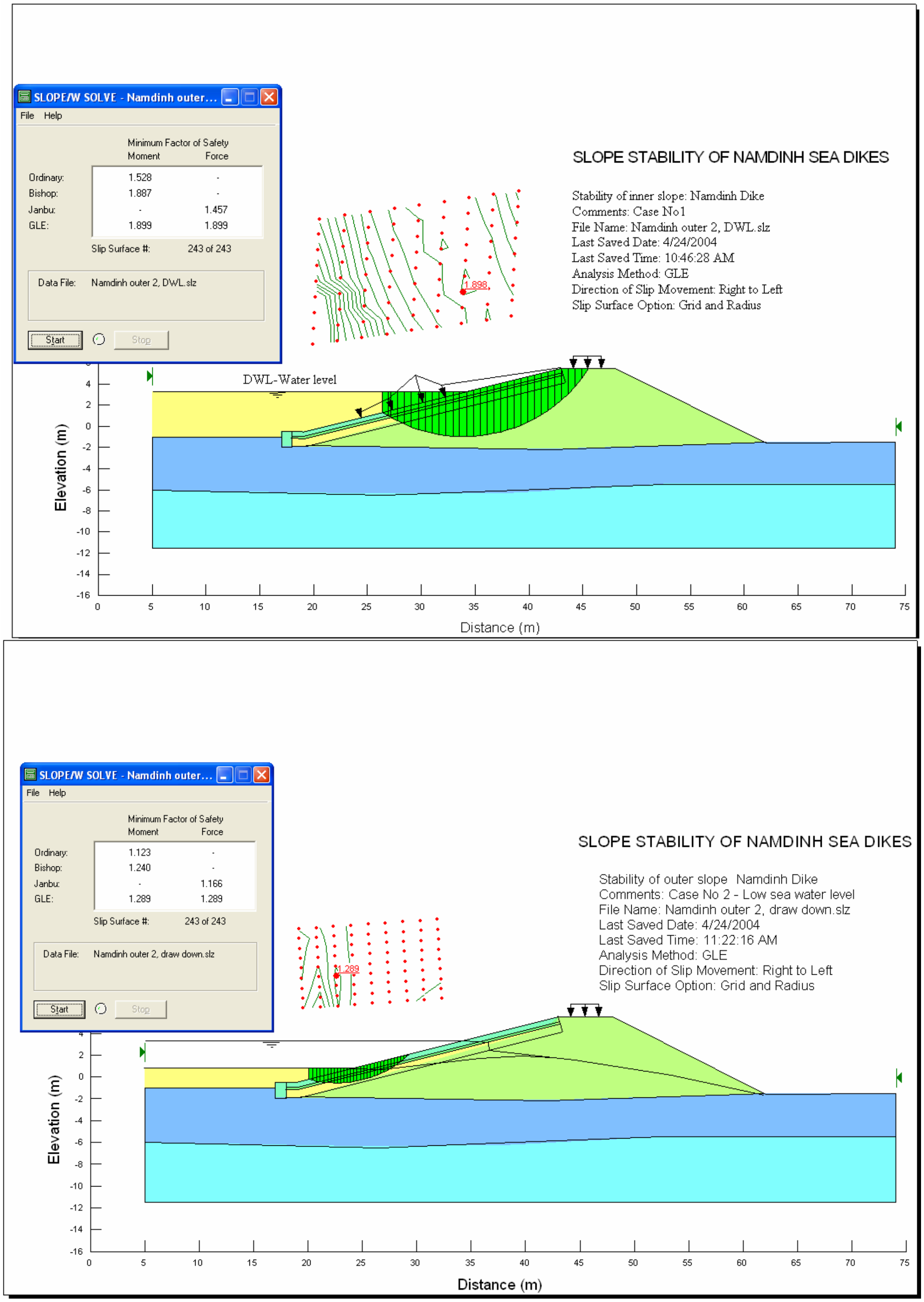


Figure 5.32: Stability of outer slope – GLE and Bishop Methods

In case of inner slope, determination of stability was done for the case of the highest sea water level (DWL=+3.3m) and the lowest water level at land side. The result is shows in Figure 5.33. The safety factor of inner slope is **1.150** by Bishop's method and **1.208** by GLE method.

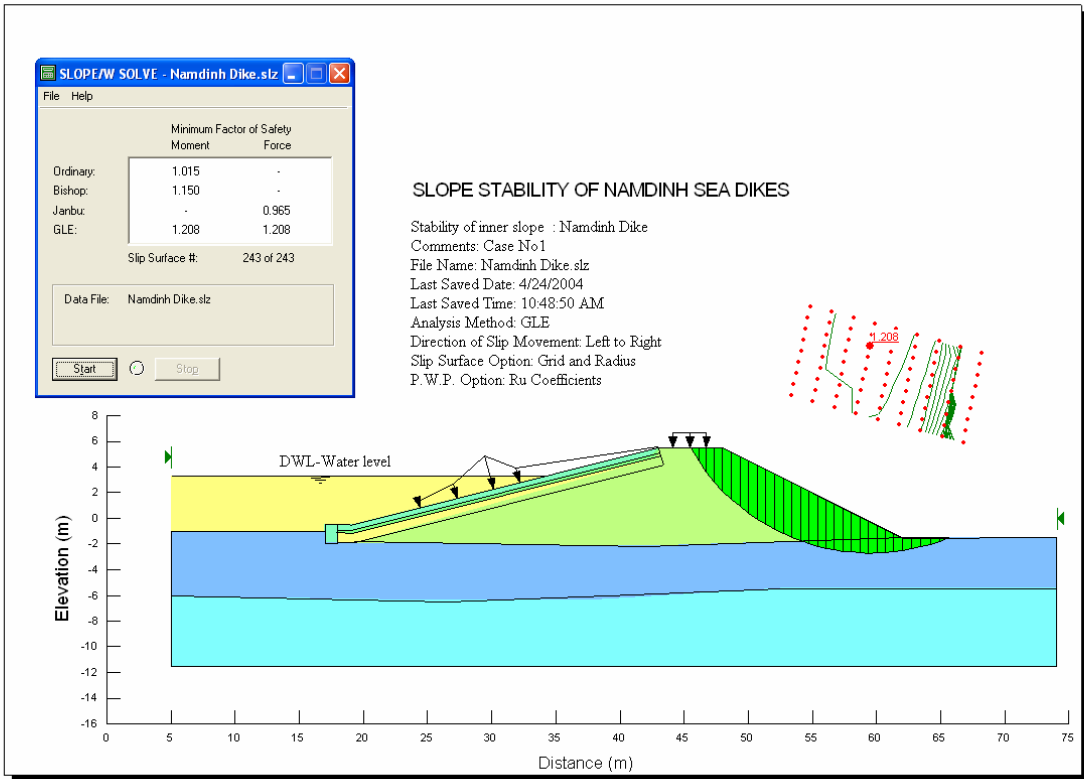


Figure 5.29: Stability of inner slope – GLE and Bishop methods

5.2.3.7 Piping.

Piping occurs when the following three conditions must be fulfilled (see TAW 2000, Technical report on soil retaining structures):

a. Condition for sand boil: Flow gradient large enough for uplift of soil, thus :

$$i_{\text{upward}} * SF \geq \frac{\rho_s - \rho_w}{\rho_w} \quad \text{(a)}$$

b. Condition for forming pile in sand: There must be a "roof" of clay, concrete or otherwise.

c. Ground water head must be larger then critical hydraulic head (reduced by a safety factor)

$$\Delta H_{strenght} \leq SF * \Delta H_{loading} (= SF * (\Delta H - 0.3d)) \qquad \text{(c)}$$

Applying for the case of Namdinh dikes (with safety factor of 1.20)

Checking condition (a): According to results of groundwater analysis, the maximum upward hydraulic gradient is 0.5. Condition (a) becomes

$$i_{\text{upward}} * SF \geq \frac{\rho_s - \rho_w}{\rho_w} \Longleftrightarrow 0.6 = 0.5 * 1.20 \geq \frac{19-10}{10} = 0.9 \rightarrow Not\_satisfy$$

Checking condition (b): The difference factor for permeability between body soil and sub-surface soil is 7.5, therefore it should be consider that there is not any "roof" for forming a pile in sub-surface layer.

Checking condition (c):

$\Delta H_{strength}$ is critical hydraulic head, can be determined according to Bligh,

$$\Delta H_{strength} = \frac{L}{C_{creep}} = \frac{46m}{15} = 3.1 \text{ m}$$

In which

C_{creep} is material constant, determined accord. Bligh, with an assumption of sub-surface soil has the characteristic of partly sand clay, so selected C_{creep} =15.

L is seepage length, from entry point to exit point of the first flow line; in this case, assuming that the seepage length is from the lowest point of outer slope to that of inner slope of the dike.

ΔH is hydraulic head over the dike.

$\Delta H = 3.3 - 0.25 = 3.05$m

And therefore the condition (c) for piping is not fulfilled.

Finally, conclusion can be made that the piping does not occur at Namdinh sea dikes.

CHAPTER 6 PROBABILISTIC ASSESSMENT OF THE SAFETY OF NAMDINH SEA DIKES

The deterministic approach for ad-judgement of safety of sea dikes in Vietnam with case study at Namdinh sea dikes were introduced in chapter 5. The result of these analyses provided the certain safety factor corresponding to each possible failure mechanism.

In this chapter the probabilistic calculations are applied intending to investigate 'how safe the dikes are' in term of providing failure probability of whole dike system, which is contributed by all possible failure mechanism of the elements. However, due to the limited time and lack of data, the investigation of safety of the dikes will be related to four possibly dominant failure mechanisms as Overtopping; Instability of slope protection; Macro-instability; Piping; and the investigation is applied for one representative section at HaiTrieu committee. The probabilistic approach used in this study is at level II which considers that most of all the stochastic variables of reliability function are followed the normal distribution.

6.1 Introduction

The basic of the deterministic approach are the so-called design values for the loads and the strength parameters. Loads for instance are the design water level and the design significant wave height. Using design rules according to codes and standards it is possible to determine the dimensional geometrical parameters of the flood defences such as the dikes (the shape and the height of the cross section). These design rules for the sea dikes are, in general, based on limit states of the flood defence system's elements, such as overtopping, erosion, instability, piping and settlement.

In deterministic approach it is assumed that the structure is safe when the margin between the design value of the load and the characteristic value of the strength is large enough for all limit states of all elements. Therefore the safety level of the protected area is not explicitly known when the flood defence is designed according to the deterministic approach.

The most important shortcomings of the deterministic approach are (Vrijling, 1998):

- The fact that the failure probability of the system is unknown.
- The complete system is not considered as an integrated entity.
- The length of flood defence does not affect the design (not take into account the length effect). In the deterministic approach the design rules are the same for all the sections independently of the number of sections. It is however intuitively clear that the probability of flooding increases with the length of the flood defence.
- With the deterministic design methods it is impossible to compare the strength of different types of cross-sections such as dikes, dunes and structures like sluices and pumping stations.
- The deterministic design approach is incompatible with other policy fields like for instance the safety of industrial processes and the safety of transport of

dangerous substances. The magnitude of the damage or loss has no influence on the design of dike.

- The actual probability of inundation of the region protected by the dike system is not known.

A fundamental difference with the deterministic approach is that the probabilistic design methods are based on an acceptable frequency or probability of flooding of the protected area.

The probabilistic approach results in a probability of failure of the whole flood defence system taking account of each individual cross-section and each structure. So the probabilistic approach is an integral design method for the whole system.

The accepted probability of flooding is not the same for every polder or floodplain. It depends on the nature of the protected area, the expected loss in case of failure and the safety standards of the country. For this reason accepted risk is a better measure than an accepted failure probability because risk is a function of the probability and the consequences of flooding.

The most general definition of risk is the product of the probability and a power of consequences: *Risk = (probability) * (consequence)*^{n}. The power n is depend on the situation of the system, n=1 is a natural risk approach and implies the calculation of expected value while n>1 denotes the risk aversion.

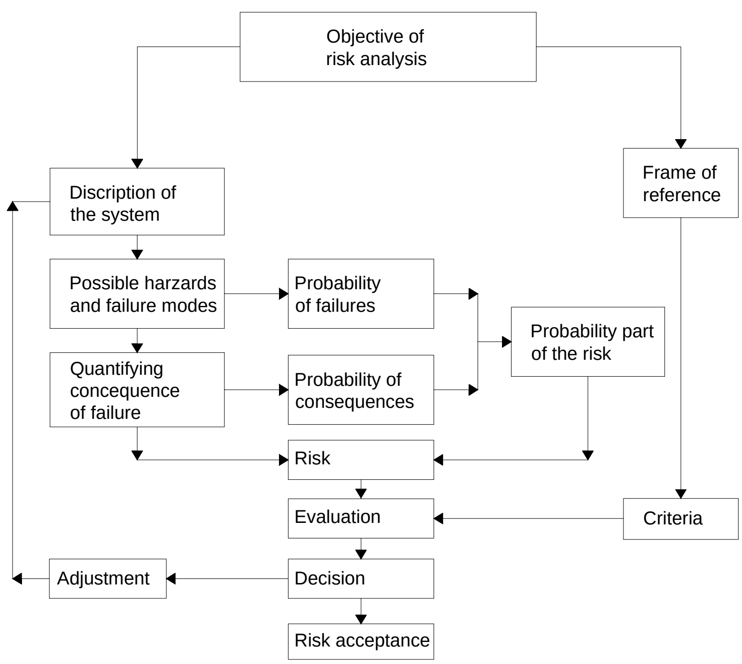


Figure 6.1: Frame work of risk analysis (see CUR 141, 1990)

Figure 6.1 shows the elements of risk analysis in the probabilistic approach. First of all the flood defence system has to be described as a configuration of elements such as dike sections, sluices and other structures. Then an inventory of all the possible hazards and failure modes must be made. This step is one of the most important of the analysis because missing a failure mode can seriously influence the safety of the design. The next step can be the quantifying of the consequences of failure for all possible ways of failure.

The failure probability and the probability of the consequences form the probability part of the risk. When the risk is calculated the design can be evaluated. For this criteria must be available such as a maximum acceptable probability of a number of casualties or the demand of minimizing the total costs including the risk. For determining the acceptable risk it needs to refer to a frame of reference. This frame of reference can be the national safety level aggregating all the activities in the country. After the evaluation of the risk one can decide to adjust the design or to accept it with the remaining risk.

6.2 General background of probabilistic calculation

The probabilistic calculation of ascertaining the probability of failure is based on the reliability function. The reliability function Z is established with regard to the limit state considered, in such a way that negative value of Z corresponds to failure and positive values to non-failure (see Figure 6.2). The probability of failure thus be represented as $P\{Z<0\}$. The reliability function is a function of a number of stochastic variables.

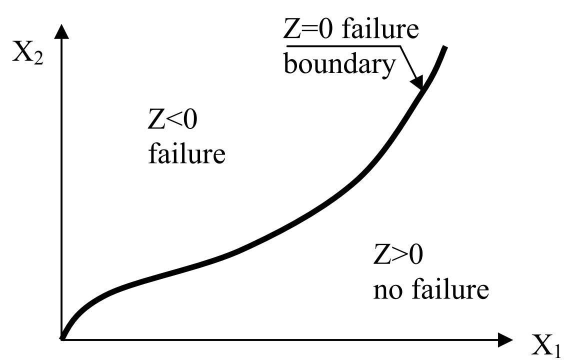


Figure 6.2: Definition of a failure boundary Z=0

There are various techniques available for determining the probability of failure for a given reliability function and given statistical characteristics of the basic variables. For classifying these techniques the following international-agreed levels are to be distinguished:

Level III: Comprises calculations in which the complete probability density functions of the stochastic variables are introduced and the possibly non-linear character of the reliability function is exactly taken into account.

Level II: Comprises a number of approximate methods in which the problem is linearized and all probability density functions are replaced by probability density functions of normal distributions.

Level I: Comprises calculations based on characteristic values and (partial) safety factors or safety margins.

The various levels and their interrelation for flood defences, in detail, can be seen at CUR/TAW 141, 1992.

6.3 Probabilistic assessment of the safety of Namdinh sea dikes

6.3.1 General reliability function and failure probability calculation

Calculation at level II is applied in this study to estimate the probability of failure of Namdinh sea dikes. The general form of reliability function Z=R-S is considered. In which R and S are functions of strength and load respectively and both considered following normal distribution. This implies that the statistical parameter of reliability function Z can be obtained through:

$$\mu(Z) = \mu(R) - \mu(S) \qquad (6\text{-}1)$$

$$\sigma^2(Z) = \sigma^2(R) + \sigma^2(S) \qquad (6\text{-}2)$$

The probability density function of Z given by:

$$f(Z) = \frac{1}{\sqrt{2\pi\sigma^2}} e^{\frac{-(Z-\mu)^2}{2\sigma^2}} \qquad (6\text{-}3)$$

The probability distribution function (cumulative distribution function) of Z:

$$F_Z(a) = \int_{-\infty}^{a} f_z(X)dX = \Phi_N\left(\frac{a-\mu}{\sigma}\right) \qquad (6\text{-}4)$$

The probability of failure of the element which has the reliability function Z is $F_Z(a=0)=P(Z<0)$:

$$P\{Z<0\} = \int_{-\infty}^{0} f_z(X)dX = \Phi_N(-\beta) \qquad (6\text{-}5)$$

where:

β is reliability index; $\beta = \frac{\mu}{\sigma}$

$\Phi_N(-\beta)$ Standard normal distribution for the variable β

In general Z in equation (6-5) will be a function of more than two variables. These variables do not have to be normally distributed and Z does not have to be linear. Only if Z is a linear function and all variables are normally distributed (and independent) the second equation in (6-5) is indeed equality and not an approximation.

Z may be a function of n stochastic variables X1, X2,...,X , as both the "load", S, and the "strength", R, may depend on more than one variable. In order to perform a level II calculation, the variables X1, X2,...,X have to be independent and it must be possible to linearize the reliability function Z in all point of Z. Suppose the reliability function, Z, fulfils the requirement and the variables X are all normally distributed and independent.

It is supposed that the reliability function can be linearized , so tangent plane in a point on it surface can be expressed by a first order Taylor expansion:

$$Z_{lin} = Z(X_1^*, X_2^*, ...X_n^*) + \sum_{i=1}^{n} (X_i - X_i^*) * \left(\frac{\partial Z}{\partial X}\right)_{X_i = X_i^*} = 0 \qquad (6\text{-}6)$$

Z_{Lin} = Linearized reliability function of Z in $\{X_i^*\}$

$\left(\frac{\partial Z}{\partial X}\right)_{Xi=Xi*}$ = partial derivative of Z with respect to X_i, evaluate in $X_i = X_i^*$

The mean value and standard deviation of Z_{Lin} are:

$$\mu(Z_{lin}) = Z(X_1^*, X_2^*, ...X_n^*) + \sum_{i=1}^{n} (\mu_{Xi} - X_i^*) * \left(\frac{\partial Z}{\partial X}\right)_{X_i = X_i^*} \quad (6\text{-}7)$$

$$\sigma^2_{(Z_{lin})} = \sum_{i=1}^{n} \sigma^2_{Xi} * \left(\frac{\partial Z}{\partial X}\right)^2_{X_i = X_i^*} \quad ... \quad (6\text{-}8)$$

The probability of failure and reliability index, as definition in Figure 6.3, given by:

$$P\{Z < 0\} = \int_{-\infty}^{0} f_z(\xi) d\xi = \Phi_N(-\beta) \quad (6\text{-}9)$$

where $\beta = \frac{\mu(Z_{Lin})}{\sigma(Z_{Lin})}$

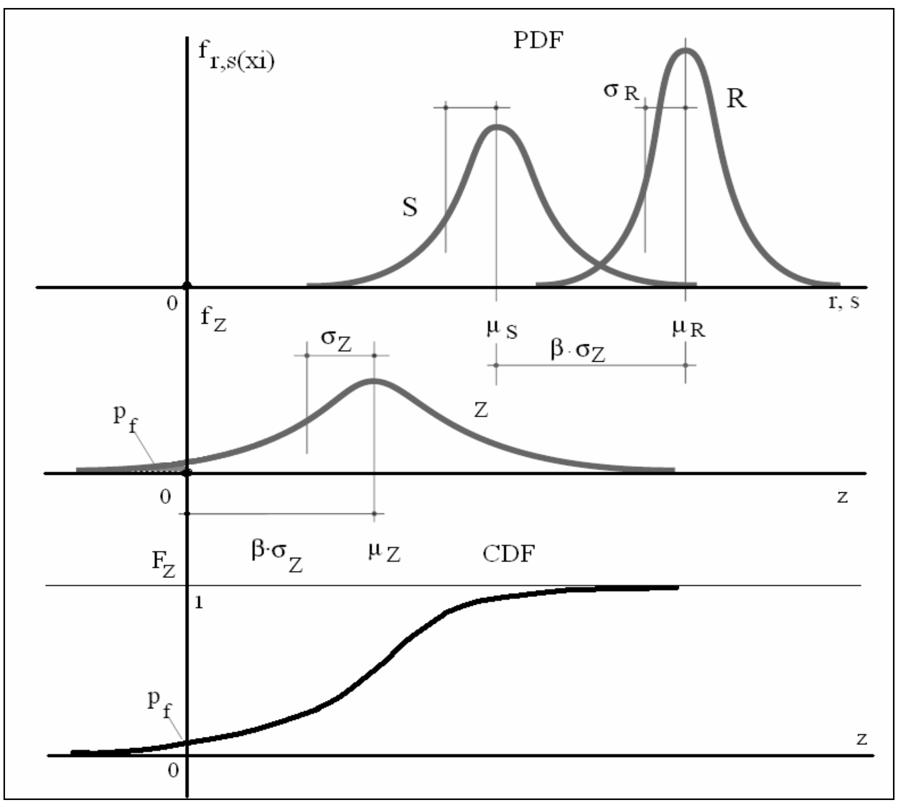


Figure 6.3: Definition of probability of failure and reliability index.

If mean values $X_1^* = \mu_{(Xi)}, \ldots, X_n^* = \mu_{(Xn)}$ are situated, a so called mean value approximation of the probability of failure is obtained. If the failure boundary is nonlinear, a better approximation can be achieved by linearization of the reliability function in the Design Point. The Design Point is only defined if the variables are normally distributed (or are transformed to normal distributed variables). The Design Point is defined as the point on the failure boundary in which the Joint (normal) probability density is maxima.

The design point is given:

$$X_i^* = \mu_{X_i} - \alpha_i . \beta . \sigma_{X_i} \tag{6-10}$$

$$\alpha_i = \frac{\sigma(Xi)}{\sigma(Z_{Lin})} * \frac{\partial Z}{\partial X_i} \quad \text{(Influence factor of variable number i)} \tag{6-11}$$

6.3.2 Statement of the problem

The possible failure mechanisms for the sea dikes were indicated in Chapter 3 and 4. In this section the failure of Namdinh dikes consist of overtopping, instability of slope protection, macro-instability of inner and outer slope, and piping.

The failure of the dike system will not occur if all dike sections, which are consecutive and forming the dike system, are stable.

For a dike section the failure may occur since there will present any failure mechanism.

The fault tree of the dike, in this case, is shown on Figure 6.4.

Due to the limitation as said earlier the failure probability of Namdinh sea dike is estimated only for one section at HaiTrieu committee. For the whole system of Namdinh sea dikes further study on probabilistic calculation and risk analysis should be taken and much more data need to be provided.

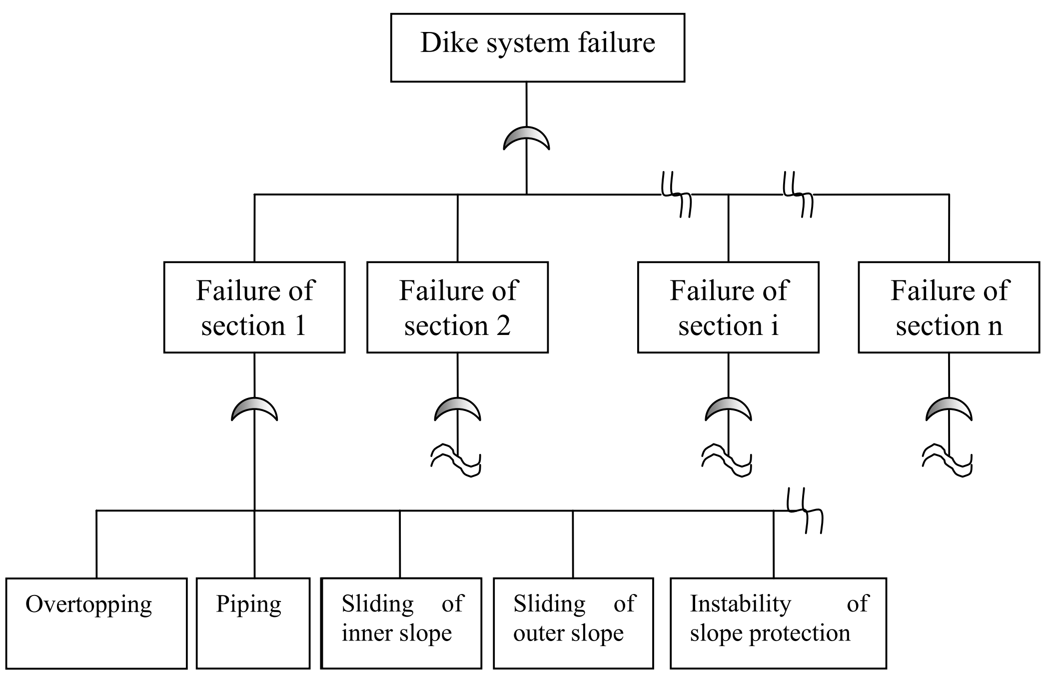


Figure 6.4: Fault tree of Namdinh sea dike.

6.3.3 Probability of failure mechanism

6.3.3.1 Overtopping

Reliability function of overtopping failure mode

The failure mechanism of overtopping occurs when the sea water level (Z_{max}) exceed the design crest level (Z_c) of the dike (See also *Section 4.2.1.1*). The reliability function of overtopping becomes:

$$Z = Z_c - Z_{max} \tag{6-12}$$

Where:

Z_c : Crest level of the dike

Z_{max}: Maximum water level acts on the dike (including wave run-up level)

The failure presents when $Z<0$ therefore the probability of overtopping failure mode is $P(Z<0)$.

Calculation procedure

Crest level: Crest level is supposed normal distribution. The mean value is from deterministic calculation. The standard deviation is 0.20 m which is considered an error of construction process.

Maximum water level:

$$Z_{max}=DWL+\text{Run-up level} \tag{6-13}$$

The ***DWL*** is given by:

$$DWL=MHWL\ (MSL+\text{High tide}) + \text{Surge} + \text{Sea level rise} \tag{6-14}$$

(see also section 5.1.1.1)

Where :

MHWL= MSL+High tidal level which can be obtained from observation. Along Namdinh coast the observation of water level is at Vanly station which gives the data of 16 years. Based on that statistical data the distribution of MHWL can be established. According to the analysis by using BESTFIT, see attached Figure 6.5, the maximum water level is quite fitted to normal distribution with the mean value of 2.29 m and standard deviation of 0.071m.

Surge and Sea level rise are assumed to be normal distributions. The selection of mean value based on the value in deterministic calculation and standard deviations are shown in Table 6.1:

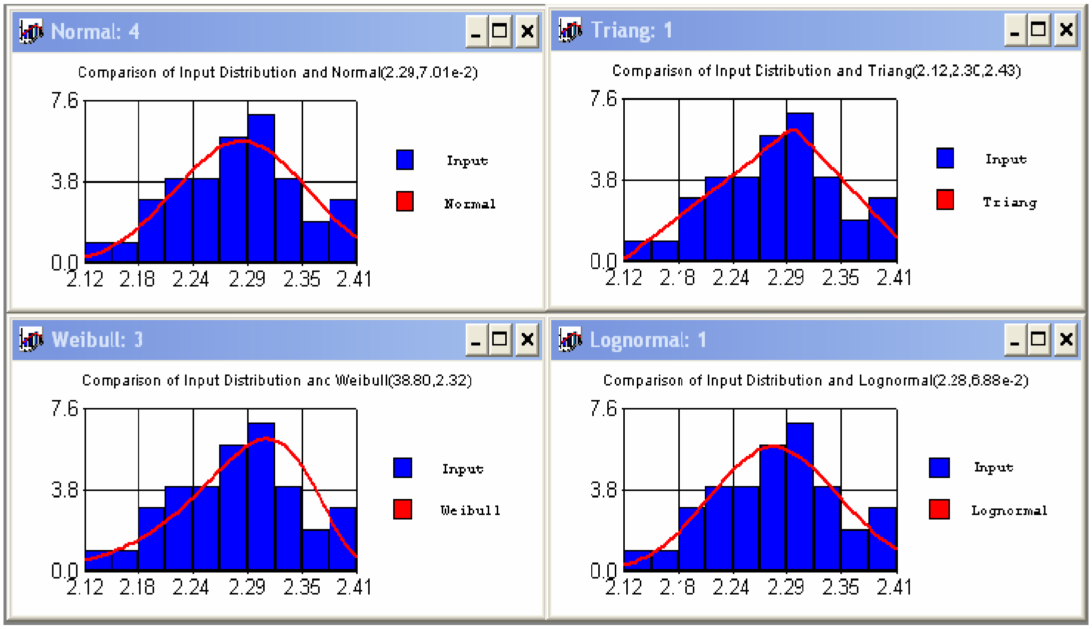


Figure 6.5: Distribution of MHWL based on statistical data by using BESTFIT

Since the distribution of stochastic components known the DWL can be determined statistically by using VaP[(1)] model, follows equation (6-14).

Table 6.1: Determination of DWL

X	*Description of variable*	*Unit*	*Type of distribution*	*Parameter (1, 2)*	
				Mean	*Std. deviation*
MHWL	High tidal level(+MSL)	m	Nor	2.29	0.071
Surge	Incr. W.level by surge	m	Nor	1.0	0.2
S.L rise	Sea level rise	m	Nor	0.1	0.05
DWL	Design water level	m	MHWL+Surge+S.L.Rise		

Run-up level ($Z_{2\%}$) :

Run up level were determined by formula in actual Vietnamese design code and Van der Meer formula in Dutch code [*Formula (4) and (7), See section 5.2.1. a*]

As said earlier due to lack of statistical wave data, therefore, these formulae are used the significant wave height at the toe of the structure, in this case, which determined by depth limited wave height, $H_s = a*d$, where a is empirical coefficient, suggested a=0.45 to 0.55; d is water depth at toe of the dike.

The depth limited wave height as a function of water depth, d, and the empirical parameter, a. The depth at the toe of the dike depends on the DWL and the level of bed near the toe. The bed level is supposed normal distributed which has the mean value as nominal measured value and standard deviation approximately equal to the considered annual maximum scour depth and error of measurement.

Once the distribution of related stochastic variables known the depth limited wave height can be determined statistically by using VaP[(1)] model, follows the given relation, see Table 6.2.

Table 6.2: Determination of Hs (Depth limited wave height)

X	*Description of variable*	*Unit*	*Type of distribution*	*Distributed parameter*	
				Mean	*Std. deviation*
MHWL	High tidal level(+MSL)	m	Nor	2.29	0.071
Surge	Incr. W.level by surge	m	Nor	1.0	0.2
S.L rise	Sea level rise	m	Nor	0.1	0.05
Z_{bed}	Level of sea bed near toe	m	Nor	nom	0.2
a	Empirical coefficient	-	Nor	0.5	0.05
d	The water depth	m	= DWL-Z_{bed} =(MHWL+Surge+S.L.Rise)-Z_{bed}		
Hs	Design wave height	m	= a*d = a*{(MHWL+Surge+S.L.Rise)-Z_{bed}}		

Since the wave height is a function of MHWL, Surge, S.L.Rise, Z_{bed} and a. then the wave run up 2% is dependent on these stochastic variables as well.

The additional stochastic variables for determination of wave run up 2% are listed in Table 6.3 and 6.4, depends also on the applied formulae.

Wave run up formula in Vietnamese code:

$$\Delta Z_{p\%} = \frac{K_\Delta . K_w K_p}{\sqrt{1+m^2}} \sqrt{Hs.Ls} \qquad (6\text{-}14)$$

Ls is local wave length, Ls=$T_m\sqrt{g * d}$, others see S*ection 5.2.1.a*

Table 6.3: Additional stochastic variables for determination of $Z_{2\%}$ by Vietnamese code

X	*Description of variable*	*Unit*	*Type of distribution*	*Distributed parameter*	
				Mean	*Std. deviation*
K_Δ	reduction factor for slope roughness	m	Nor	Nom-Depend on slope protection.	0.05
K_w	coefficient of wind effect	-	der	1	-
K_p	transformed coefficient % of wave exceedance	m	der	1.65	-
m	Cotangent of slope angle	-	Nor	4	0.15
T_m	Wave mean period	s	Deter	nom (8.5)	

Wave run up formula in Dutch code (Van der Meer's formula, TAW 2002):

$$\frac{Z_{2\%}}{H_{m0}} = 1.65\gamma_b\gamma_f\gamma_\beta\xi_o = 1.65\gamma_f\gamma_\beta\xi_{eq} \quad \text{for } 0.5 \le \xi_{eq} \le 1.8; \qquad (6\text{-}15)$$

$$\frac{Z_{2\%}}{H_{m0}} = \gamma_f\gamma_\beta(4.0 - 1.5/\sqrt{\xi_o}) \quad \text{for } 1.8 \le \xi_{eq} \le 8 \text{ to } 10 \qquad (6\text{-}16)$$

where ξ_{op} is breaker parameter, given by : $\xi_{op} = \frac{\tan\alpha}{\sqrt{\frac{2\pi H_s}{gT_p^2}}}$; for these others see S*ection 5.2.1.a*

In this formula model factors (1.65, 4.0 and 1.5) are considered as these stochastic parameters. The mean values are taken by the factors in the equation and standard deviations in relation of the mean values with variation coefficient of 0.07 (mean divided by standard deviation is equal to 0.07-TAW 2002). The other stochastic variables are given in Table 6.4. For given case of Namdinh sea dikes formula (6-15) is applied.

Table 6.4: Additional stochastic variables for determination of $Z_{2\%}$ by Dutch code

X	*Description of variable*	*Unit*	*Type of distribution*	*Distributed parameter*	
				Mean	*Std. deviation*
γ_f	reduction factor for slope roughness	m	Nor	Nom-Depend on slope protection.	0.05
γ_b	reduction factor for a berm	-	Deter	1	-
γ_β	reduction factor for wave angle	m	nor	0.95	0.05
m	Cotangent of slope angle	-	Nor	4	0.15
T_{op}	Wave mean period	s	Deter	10.2	-
6.5	Model factor	-	nor	6.5	0.07*6.5=0.46

Limit state function: $Z_{ovetopping.}=Z_c-Z_{max}=Z_c-(MHWL+Surge+S.L.Rise+Z_{2\%})$

Since the distribution of all stochastic variables in the reliability function are known it is possible to calculate the failure probabilities. In this report the VaP(1) model is used to determine the failure probabilities of defined reliability function *(Appendix 3).*

(1) The ***VaP*** model was developed by Institute of Structural Engineering IBK-ETH Zurich, Switzerland. The program lends itself to reliability analysis with reliability function (limit state function) and may also be used in a much wider context when evaluating the influence of variables for a problems and establishing new variable which is contributes by some other basic variables.

The failure probabilities of overtopping mechanism are investigated for three cases at the present situation: existing dike's crest level; new determined crest level by applying Vietnamese code; new determined crest level by applying Dutch code. The failure probabilities of the dikes due to overtopping, according to FORM analysis (First Order Reliability Method) by using VaP, are shown in Table 6.5.

Table 6.5: Failure probabilities of the dikes due to overtopping

Type of slope protection	*Parameter*	*Unit*	*Existing dikes*	*New required dike design by applying deterministic approach*	
				Vietnamese Code	*Dutch Code*
Riprap revetment	Design crest level	m	5.50	6.60	7.60
	Failure probability	-	0.474	0.0474	0.0501
	Reliability index	-	0.0646	1.67	1.64
Concrete placed block	Design crest level	m	5.50	7.60	8.75
	Failure probability	-	0.632	0.0464	0.0201
	Reliability index	-	-0.338	1.68	2.05

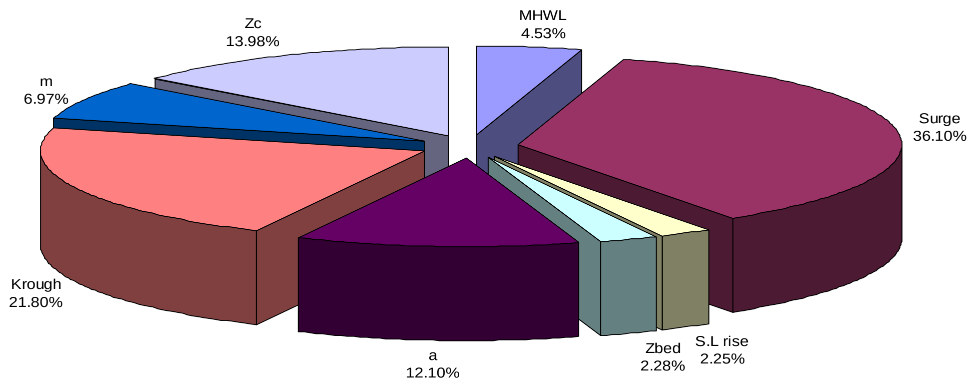
The result of FORM analysis gives also the value of reliability index β and the values of, α_i, factor of influence of stochastic variables (*Appendix 3*).

Influences of each stochastic variable to the total variance of reliability function are shown in Table 6.6 and Figure 6.6. It shows that the {(MHWL) + Surge} contributes the most to the overtopping failure mode while the sea level rise is the less contributed parameter. On the other hand, the model factor and roughness coefficient also contributes considerably to the failure mode. It means that selection of type of slope protection and its model factor will take a considered part to overtopping.

Table 6.6 Contribution of X_i to overtopping failure mode

Dutch Code, run up by Van der Meer 2002					*Vietnamese Code, run up by Chinese-formula (4)*				
No.	*Xi*	α_i	$(\alpha_i)^2$	*% contribution*	*No.*	*Xi*	α_i	$(\alpha_i)^2$	*% contribution*
1	MHWL	0.159	0.0253	3.06	1	MHWL	0.213	0.0454	4.54
2	Surge	0.449	0.2016	24.42	2	Surge	0.601	0.3612	36.12
3	S.L rise	0.112	0.0125	1.54	3	S.L rise	0.15	0.0225	2.25
4	Z_{bed}	-0.272	0.0740	7.62	4	Z_{bed}	-0.151	0.0228	2.28
5	a	0.295	0.0870	10.63	5	a	0.348	0.1211	12.11
6	K_{rough}	0.523	0.2735	32.85	6	K_{rough}	0.467	0.2181	21.81
7	m	-0.236	0.0557	6.97	7	m	-0.264	0.0697	6.97
8	Zc	-0.315	0.0992	12.90	8	Zc	-0.374	0.1399	13.99
9	1.65	0.414	0.1714		Total			1.0	100
Total			1.0	100					

VIETNAMESE CODE- RUN UP BY CHINESE FORMULA



DUTCH CODE-RUN-UP BY VAN DER MEER 2002

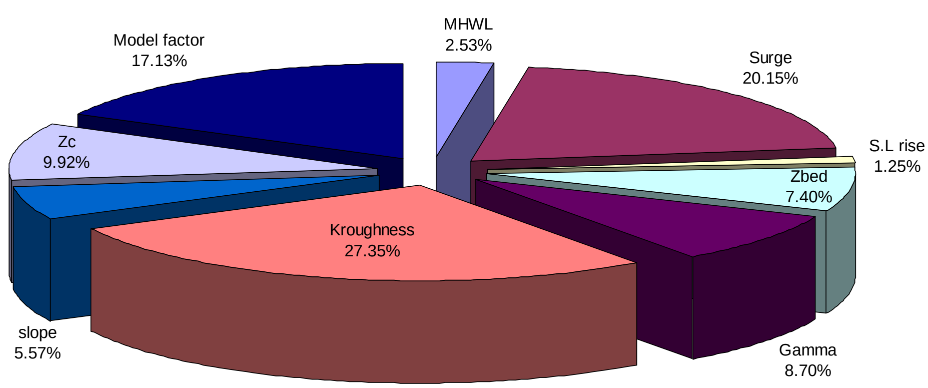


Figure 6.6: Contribution of variables to overtopping failure mode.

6.3.3.2 Instability of armour layers of revetment

Reliability function of stability of armour layers

The reliability function of stability of armour layer of revetment depends on each type of slope protections and which stability formula was applied. The common form of reliability function is:

$$Z = (H_s/\Delta D)_R - (H_s/\Delta D)_S \qquad (6\text{-}17)$$

where Δ:is relative density of applied material; D is the size of elements

Calculation procedure

+) Term $(H_s/\Delta D)_R$

For existing dikes, the $(H_s/\Delta D)_R$ is the value which applied for slope protection of Namdinh sea dikes.

For the actual situation of boundary conditions of Namdinh dikes the $(H_s/\Delta D)_R$ is the value which was calculated by deterministic method.

+) $(H_s/\Delta D)_S$: is determined by difference formulae and design codes. When applying Vietnamese design code: both riprap (rock) and block revetment the Pilarczyk's formula was used. While applying Dutch design code: the Van der Meer's formula was used for rock revetment; the Pilarczyk's formula was applied for block revetment

Wave height Hs: The depth limited wave height, again, is used. However, for stability calculation of slope protection, it is necessary to know the sensitivity of variable Hs contributed to the instability of elements of slope protection. For this reason, instead of describe Hs in the reliability function (6-17) as the relation of water level, bed level and coefficient a, an approximation of wave height distribution is made by establishing wave height distribution function.

The applied wave height in Van der Meer formula for shallow foreshore condition is $H_{2\%}$. In this case with relatively very shallow foreshore, therefore, assume that $H_{2\%}=H_s=a*d$.

The wave height distribution function is simulated in VaP based on the relation:

$Hs= a*\{(MHWL+Surge+S.L.Rise)-Z_{bed}\}$. The related stochastic variable are selected exactly the same in Table 6.2. Applied Monte Carlo analysis in VaP, one can obtain that the histogram of wave height is approximately lognormal distribution (see *Appendix 3*).

Since Z_{bed} is different for design situation and present situation it leads to the difference in the design wave height between these situations, see Table 6.7. The others related stochastic variable as in Table 6.2.

Table 6.7: Approximation of wave height distribution

Parameter	*Type of distribution*	*Design situation*		*Present situation*	
		Mean	*Std. deviation*	*Mean*	*Std. deviation*
Z_{bed}(m)	nor	-0.50	0.2	-1.30	0.2
H_s (m)	Lognor	1.90	0.245	2.3	0.269

The failure probabilities of instability mechanism are investigated for three cases:

- existing dikes;
- new determined rock size by applying deterministic design in Vietnamese code;
- new determined rock size by applying deterministic design in Dutch code.

The stochastic variables for reliability functions of slope protection are in Table 6.8. The failure probabilities of instability mechanism are shown in Table 6.9

Table 6.8: Stochastic variables of failure probabilities of slope protection instability

X_i	*Description of variable*	*Unit*	*Type*	*Distributed parameter*	
				Mean	*Std. deviation*
Reliability function based on Pilarczyk's Formula for placed block revetment **$Z=\{\phi*\Delta*D\}-H_s*(\tan\alpha/SQRT(S_0))^b/\cos\alpha$**					
Hs	Design wave height	m	LogNor	Table 6-7	Table 6-7
$\tan\alpha$	Tangent of slope angle	-	Nor	0.25	0.018 (error 1^o)
S_0	Wave steepness	-	Deter	0.02	
$\cos\alpha$	Cosine of slope angle	-	Nor	0.97	0.05 (error 1^o)
Δ	Relative density of concrete	-	Nor	1.4	0.05
ϕ	Empirical factor -Pilarczyk	-	Nor	5	0.5
b	Power factor	-	Nor	0.65	0.15
D	Required block thickness	m	Deter	nom	
Reliability function based on Van der Meer's formula for rock revetment **$Z=\{8.7*P^{0.18}*(S/N^{0.5})^{0.2}*(\tan\alpha/SQRT(S_0))^{-0.5}\}-\{H_s/\Delta/D\}$**					
Hs	Design wave height	m	LogNor	Table 6-7	Table 6-7
N	Number of storms	-	Deter	7000	
P	Permeability factor	-	Nor	0.2	0.05
S	Initial damage level	-	Deter	2	
Δ	Relative density of rock	-	Nor	1.6	0.1
$\tan\alpha$	Tangent of slope angle	-	Nor	0.25	0.018 (error 1^o)
S_0	Wave steepness	-	Deter	0.02	
D	Required rock size	m	Deter	Nom	
8.7	Model factor	-	nor	8.7	0.065*8.7=0.5655

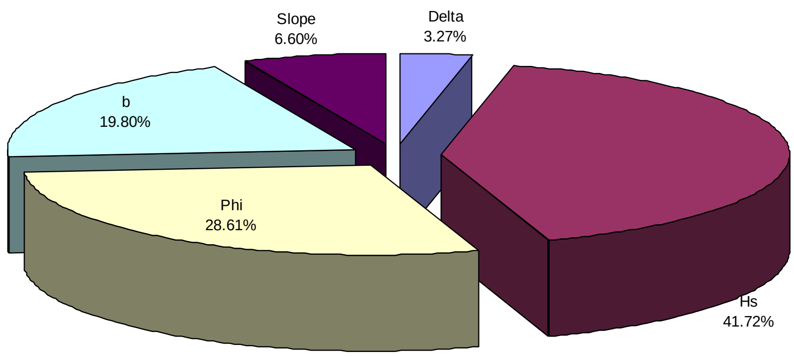
Table 6.9: Failure probabilities of the dikes due to instability of slope protection

Type of slope protection	*Parameter*	*Unit*	*Existing dikes*	*New required dike design by applying deterministic calculation*	
				Vietnamese Code	*Dutch Code*
Riprap revetment	Applied rock size	m	0.45	0.89	0.86
	Failure probability	-	0.473	0.0157	0.0274
	Reliability index	-	0.0671	2.15	1.92
Concrete placed block	Applied block size	m	0.5	0.75	0.7
	Failure probability	-	0.132	0.0123	0.0288
	Reliability index	-	1.11	2.25	1.9

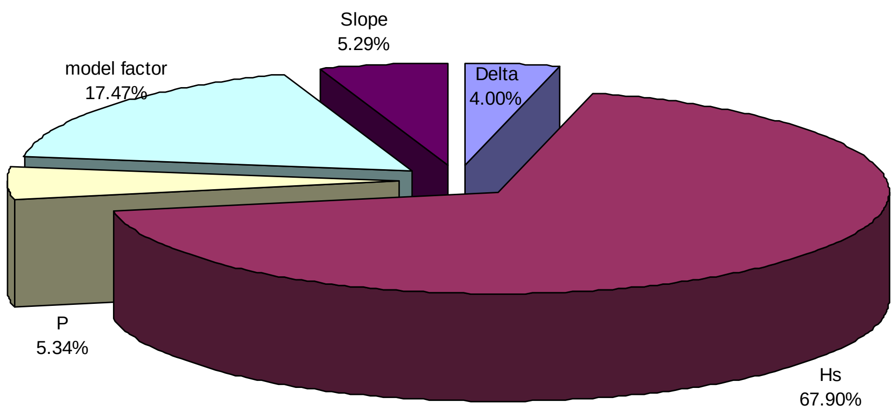
The contribution of each related stochastic variable to instability of armour layer is in Table 6.10 and Figure 6.10. As the result, the wave height gives the most influence to the probability of failure mode. When applying both formulae the model factor (model factor ϕ,b of Pilarczyk's formula, factor 8.7 of Van der Meer's formula) are also very important influence of the result.

Table 6.10 Contribution of related stochastic variable to instability of armour layer

Failure of armour layer, Pilarczyk's criteria					*Failure of armour layer, Van der meer's criteria*				
No.	Xi	α_i	$(\alpha_i)^2$	*% contribution*	*No.*	*Xi*	α_i	$(\alpha_i)^2$	*% contribution*
1	Delta	-0.181	0.033	3.28	1	Delta	-0.2	0.040	4.00
2	Hs	0.646	0.417	41.73	2	Hs	0.824	0.679	67.90
3	Phi	-0.535	0.286	28.62	3	P	-0.231	0.053	5.34
4	b	0.445	0.198	19.80	4	model	-0.418	0.175	17.47
5	Slope	0.257	0.066	6.60	5	Slope	0.23	0.053	5.29
	Total		1.0	100		Total		1.0	100



BASED ON PILARCZYK'S FORMULA



BASED ON V.D.MEER'S FORMULA

Figure 6.10: Contribution of related stochastic variable to instability of armour layer.

6.3.3.3 Piping

Piping occurs under the dike due to the erosive action of seepage flow which causes the continuous transport of soil particles. The failure mechanism of piping was described in ***Section 4.2.2.3*** (see also CUR/TAW 1990).

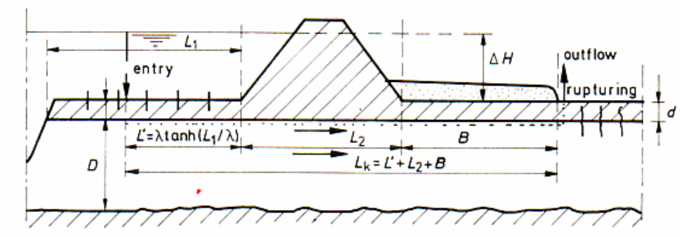


Figure 6.11: Piping at a dike (CUR 141, 1990)

The failure mechanism of piping occurs when two conditions must be satisfied:
- The clay layer under the dike must be ruptured (1)
- Continuous transport of sand must take place (2)

a. Reliability function of (1)

The rupture of clay layer occurs when the water pressure caused by high water level is higher than the wet density of the clay layer. So the reliability function that follows from the first condition is:

$$Z_1 = \rho_c * g * d - \rho_w * g * \Delta H \qquad (6\text{-}18)$$

where:

ρ_c is density of the wet clay;
ρ_w is density of water;
g is gravity acceleration;
d is the thickness of the clay layer between bottom of the dike and sand layer
ΔH is the difference in water levels between sea side and inland;

b. Reliability function of (2)

Based on the Bligh's criterion in the reliability function of piping is:

$$Z_2 = m * L_t / c - \Delta H \qquad (6\text{-}18)$$

Where:

$L_t = L' + L_2 + B + d$
$c = c_B$ (constant depending on soil type, according to Bligh)
ΔH is the difference in water levels between sea side and inland;
L'; L_2; B defined as Figure 6.7.
m is a model factor, taking into account the scatter in empirical observations. It is assumed to conforming to a normal distribution with $\mu = 1.67$ and the coefficient of variation
$V = \sigma/\mu = 0.2$ (Probabilistic design of flood defences, TAW/CUR, 1990).

c. Calculation procedure

The failure probability of each piping conditions at Namdinh sea dike are determined by using VaP model. The stochastic variables are in Table 12

Table 6.12: The stochastic variables for piping conditions

Description of variable	*Not.*	*Unit*	*Type*	*Mean*	*Std. deviation*
Wet density of the top layer	ρ_c	kG/m^3	Deter	1800	
Density of water	ρ_w	kG/m^3	Deter	1031	
Thickness of the top layer	d	m	Nor	3.5	0.2 (error =5% of thickness)
Model factor	m	-	Nor	1.67	0.33
Seepage path length	L_k	m	Nor	48	5
Bligh constant	c_B	-	Deter	15	
Difference in water levels	ΔH	m	***$=DWL-Z_{inland}=\{MHWL+Surge\}-Z_{inland}$***		
High tidal level(+MSL)	MHWL	m	Nor	2.29	0.071
Incr. W.level by surge	Surge	m	Nor	1.0	0.2
Inland water level	Z_{inland}	m	Nor	0	0.5

The results of failure probabilities for these two conditions of piping are summarized in Table 6.13. The contribution of each stochastic variable to the failure is in Table 6.14 and Figure 6.12.

Table 6.13

Reliability function Z_1	Reliability function Z_2
β_1=6.72	β_2=3.21
$P(Z_1<0)= 9x10^{-12}$	$P(Z_2<0)= 6.57x10^{-4}$

Table 6.14 Contribution of the stochastic variables to failure mode of piping

Piping-Condition 2					*Piping-Condition 1*				
No.	*Xi*	α_i	α^2_i	*% contribution*	*No.*	*Xi*	α_i	α^2_i	*% contribution*
1	Lt	-0.783	0.613	61.31	1	MHWL	0.169	0.029	2.86
2	d	-0.031	0.001	0.10	2	Surge	0.477	0.228	22.75
3	MHWL	0.108	0.012	1.17	3	Z_{inland}	-0.596	0.355	35.52
4	Surge	0.303	0.092	9.18	4	d	-0.624	0.389	38.94
5	Z_{inland}	-0.379	0.144	14.36		Total		1.0	100
6	m	-0.373	0.139	13.91					
	Total		1.0	100					

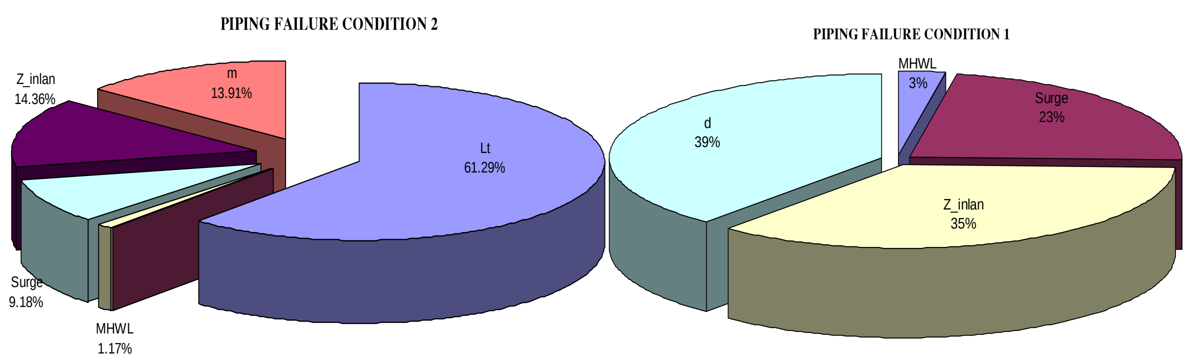


Figure 6.12: Influence of the stochastic variables to failure mode of piping

The piping failure mode occurs when (1) AND (2) are fulfilled. Therefore the probability of the failure is:

$$P_f = P\{Z=(Z_1<0 \text{ AND } Z_2<0)\}$$
$$= P\{ Z_1<0\} * P\{ Z_2<0| Z_1<0 \} \quad (6\text{-}20)$$

$P\{ Z_2<0| Z_1<0 \}$ is probability that $Z_2<0$ given $Z_1<0$

In practice determination of P{ Z_2<0| Z_1<0 } is difficult because of the complex relation and behaviour of the natural phenomena. Therefore some approximation formulae are introduced for solving this problem. One of the best known is that of Ditlevsen:

$$P\{ Z_2<0 | Z_1<0 \} \geq \max\ \{\Phi_N(-\beta_1)x\Phi_N(-\beta^*_2); \Phi_N(-\beta_2)x\Phi_N(-\beta^*_1)\} \qquad (6\text{-}21)$$

$$P\{ Z_2<0 | Z_1<0 \} \leq \Phi_N(-\beta_1)x\Phi_N(-\beta^*_2) + \Phi_N(-\beta_2)x\Phi_N(-\beta^*_1) \qquad (6\text{-}22)$$

$$\beta^* = \frac{\beta_i - \rho\beta_j}{\sqrt{1-\rho^2}} \qquad (6\text{-}23)$$

$$\rho(Z_1, Z_2) = \sum_{i=1}^{n} \alpha_i^{(1)}\alpha_i^{(2)} \qquad (6\text{-}24)$$

In which:

β is reliability index

ρ is correlation coefficient

$\Phi_N(-\beta)$ is Standard normal distribution function

α_i is the influence coefficient of variable i to Z

Table 6.15: Determination of relation parameters

	Reliability function Z_1	*Reliability function Z_1*	*$\alpha_t(1)$ *$\alpha_t(1)$*
α (MHWL)	0.169	0.108	0.018252
α (Surge)	0.477	0.303	0.144531
α (Z_inland)	-0.596	-0.379	0.225884
α (d)	-0.624	-0.031	0.019344
$\rho(Z_1, Z_2) = \sum_{i=1}^{n} \alpha_i^{(1)}\alpha_i^{(2)}$			0.408011
β	6.72	3.21	
β *	5.93	0.51	

Based on the results in Table 6.15 and equation (6-24) one can calculate ρ=0.408

Applied (6-23): β^*_1=5.93 ; β^*_2=0.51

Based on (6-21) and (6-22), the term P{ Z_2<0| Z_1<0 }, is given by:

max {Φ_N(-6.72)xΦ_N(-0.51); Φ_N(-3.21)xΦ_N(-5.93)} ≤ ***P{ Z_2<0| Z_1<0 }*** ≤ {Φ_N(-6.72)xΦ_N(-0.51)+ Φ_N(-3.21)xΦ_N(-5.93)}, then

max{10^{-9}x0.31; 0.67x10^{-3}*10^{-9}} ≤ ***P{ Z_2<0| Z_1<0 }*** ≤ 10^{-9}x0.31+0.67x10^{-3}*10^{-9}=3.1x10^{-10}

To be in safe side => P{ Z_2<0| Z_1<0 } = 3.1x10^{-10}

Hence, the probability of failure of the dike due to piping is:

$P_{\{piping\}}$= P{Z=(Z_1<0 AND Z_2<0)}= P{ Z_1<0}* P{ Z_2<0| Z_1<0 }=9x10^{-12}*3.1x10^{-10}

=3x10^{-21} ≅ **0**

This result states that the piping mechanism may not occur at Namdinh sea dike. The probability of failure of the dike due to piping can be neglected.

6.3.3.4 Sliding of dike slopes (outer and inner slopes)

The failure mechanisms of macro instability of inner and outer slopes were presented in Section 4.2.2.1. The deterministic calculations of safety of macro-stability were done in Section 5.2.3.6 by applying Generalized Limit Equilibrium (GLE) method and Bishop's method.

These deterministic slope stability analyses compute the factor of safety based on a fixed set of conditions and material parameters. If the safety factor is greater than the unity, the slope is considered to be unstable or susceptible to failure. Deterministic analyses suffer from limitation such as the variability of the input parameters are not considered and question like "How stable is the slope?" can not be answered.

Probabilistic slope stability analysis allows for the consideration of variability in the input parameters and it quantifies the probability of failure of a slope. In this study the probabilistic slope stability analysis is performed by using SLOPE/W which associates with Monte Carlo method. The input parameters of soil properties and loads are defined as stochastic variables, which can be conformed to the normal distribution function (Lumb, 1966, Tan, Donald and Melchers,1993).

Reliability function of stability of armour layer

The reliability function can be defined as: Z=SF.

The probability of failure is then defined as a probability that the safety factor is less than 1.0.

$$P_{failure}= P(Z<1)$$

Calculation procedure

The probabilistic calculation of slope stability in SLOPE/W is done by using Monte Carlo probabilistic analysis. In general, the implementation of the method is the following

- Selection of a deterministic solution procedure, in this case Bishop's method is chosen.

- Decision which input parameters are to be modelled probabilistically and representation of their variability in term of a normal distribution by given the mean value and standard deviation (see Table 6.16).

- Determination of safety factor by many times (find out the critical slip surface with $SF_{min\ min}$).

- Determination of some statistics of computed safety factors, the probability density function and the probability distribution function of the problem.

In SLOPE/W, the critical slip surface is first determined based on the mean value of the input parameters using selected limit equilibrium. Probabilistic analysis is then performed on the critical slip surface, taking into consideration the variability of the input parameters. The variability of the input parameters is assumed to be normally distributed with specified mean values and standard deviations.

During each Monte Carlo trial, the input parameters are updated based on a normalized random number. The factors of safety are then computed based on these updated input parameters. By assuming that the factors of safety are also normally distributed, SLOPE/W determines the mean and the standard deviations of the factors of safety. The probability distribution function is then obtained from the normal curve.

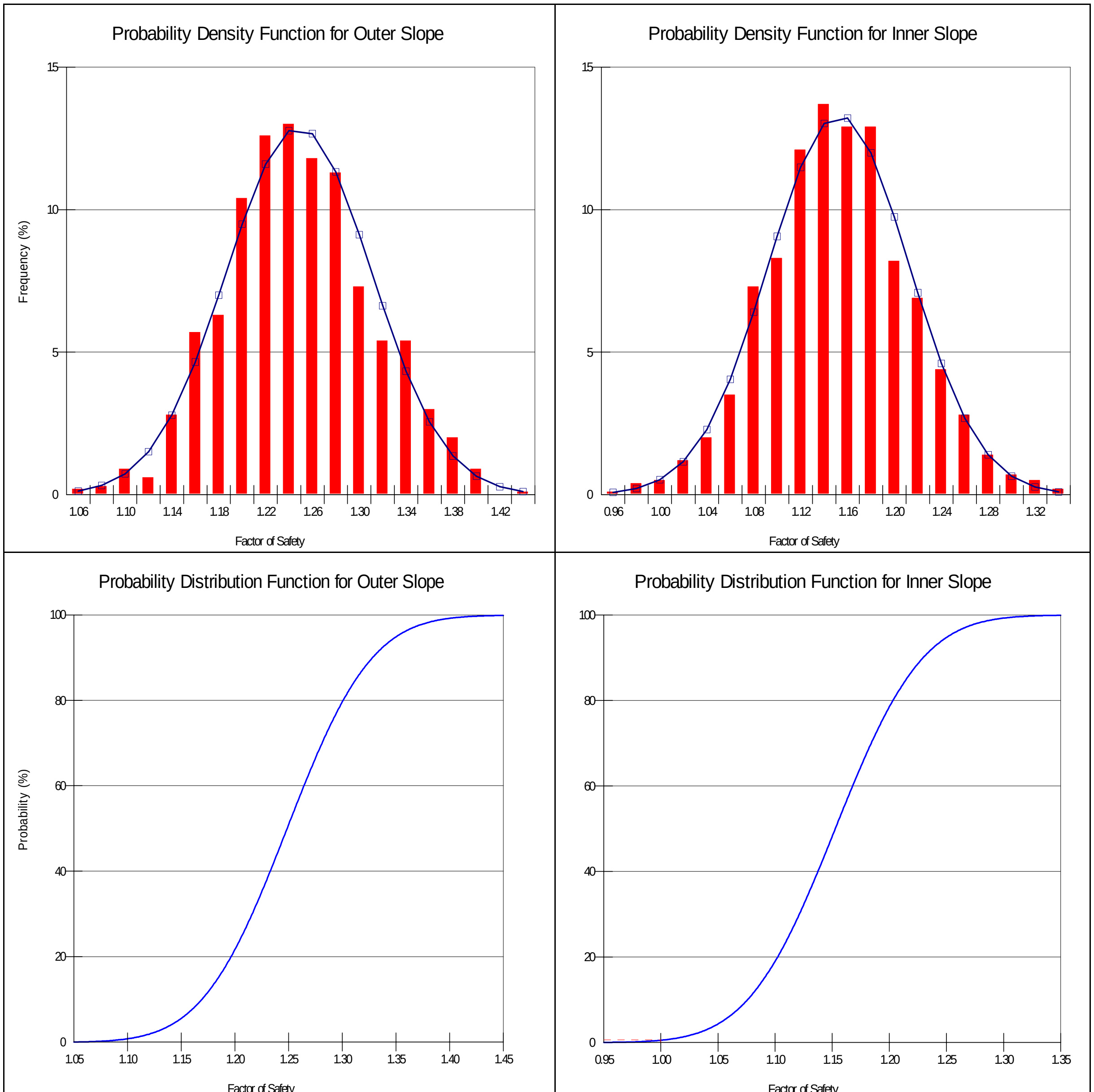
The number of Monte Carlo trials in an analysis is dependent on the number of variable input parameters and the expected probability of failure. In general, the number of required trials increases as the number of variable input increases or the expected probability of failure becomes smaller. It is normally to do thousands of trials in order to achieve an accept table level of confidence in a Monte Carlo probabilistic slope stability analysis (Mostyn and Li, 1993). The results of calculation are summarized Table 6.17 and Figures 6.13

Table 6.16: Stochastic variables of input parameters

Parameter	*Not.*	*Unit*	*Type*	*Mean*	*Std.*
Soil unit weight (unsaturated)	$\gamma_{unsat.}$	kN/m^3	Nor	nom	0.05*nom
Soil unit weight (saturated)	$\gamma_{sat.}$	kN/m^3	Nor	nom	0.05*nom
Permeability	k	m/s	Deter.	nom	
Cohesion	C	kN/m^2	Nor	nom	0.05*nom
Friction angle	φ	Degree	Nor	nom	2^0
Wave loads on outer slope	A	kN	Nor	nom	50
Transportation load on dike's crest	B	kN	Nor	100	10
nom : normative value of soil laboratory test, each has one value (see Table 5.20 also)					

Table 6.17: Summarized result of slope stability calculation

Parameter	***Outer slope***	***Inner slope***
Mean value of SF	1.1538	1.2485
Reliability Index	2.528	4
Standard Dev.	0.061	0.062
Min SF	0.98161	1.0545
Max SF	1.3416	1.4324
P (Failure) (%)	0.570860	0.003130



Outer slope	***Inner slope***

Figure 6.13: Probability and density function of slope sliding safety factor

6.3.4 Probability of dike failure

Since the probability of failure was investigated for a representative section of the dikes, one talk about the failure probability of Namdinh sea dikes this mean that the failure probability regards to only the given dike section of HaiTrieu.

The probability of dike failure now is considered as the failure probability of HaiTrieu section. For the system given in Figure 6.3, the overall failure probability of the dike can be determined as following:

$$P_{\text{dike failure}} = P\,(Z_1<0 \text{ OR } Z_2<0 \text{ OR } Z_3<0 \text{ OR } Z_4<0 \text{ OR } Z_5<0) \qquad (6\text{-}25)$$

where $\{Z_1<0 \text{ OR } Z_2<0 \text{ OR } Z_3<0 \text{ OR } Z_4<0 \text{ OR } Z_5<0\}$ denotes at least one of five failure mechanisms occur.

$Z_1<0$ denotes the failure of dike section due to overtopping

$Z_2<0$ denotes the failure of dike section due to instability of armour layer of slope protection

$Z_3<0$ denotes the failure of dike section due to instability of inner slope (Sliding mode)

$Z_4<0$ denotes the failure of dike section due to instability of outer slope (Sliding mode)

$Z_5<0$ denotes the failure of dike section due to occurring of piping

The fundamental lower and upper boundary are given by:

$$\max\{P(Z_i<0)\} \le P_{\text{dike failure}} \le \sum_{i=1}^{5} P_{Z_i}\{Z_i < 0\} \qquad (6\text{-}26)$$

in which the failure probability of failure mechanism i , $P(Z_i<0)$, was determined in previous section.

The overall failure probability of the dike, given by (6-26), are shown in Table 6.18(a,b)

Table 6.18: Overall probability of failure at Namdinh sea dike

a. For the dike with slope protection by riprap revetment:

Case	*Overtop $P(Z_1<0)$*	*Armour failure $P(Z_2<0)$*	*Sliding Outer Sl. $P(Z_3<0)$*	*Sliding inner Sl. $P(Z_4<0)$*	*Piping $P(Z_5<0)$*	*Lower bound.*	*Upper bound.*
Existing dike	0.4740	0.4730	0.000031	0.005709	3.0E-12	0.474	0.953
Deterministic design by applying Vietnamese Code	0.0474	0.0157	0.000031	0.005709	3.0E-12	0.0474	0.069
Deterministic design by applying Dutch Code	0.0501	0.0274	0.000031	0.005709	3.0E-12	0.0501	0.083

b. For the dike with slope protection by block revetment:

Case	*Overtop $P(Z_1<0)$*	*Armour failure $P(Z_2<0)$*	*Sliding Outer Sl. $P(Z_3<0)$*	*Sliding inner Sl. $P(Z_4<0)$*	*Piping $P(Z_5<0)$*	*Lower bound.*	*Upper bound.*
Existing dike	0.6320	0.1320	0.000031	0.005709	3.0E-12	0.632	0.77
Deterministic design by applying Vietnamese Code	0.0464	0.0123	0.000031	0.005709	3.0E-12	0.0464	0.064
Deterministic design by applying Dutch Code	0.0201	0.0288	0.000031	0.005709	3.0E-12	0.0288	0.055

6.3.5 Conclusion

The results analyses show that the failure probability of existing dikes in Namdinh is very high with lower probability boundary of about 47.4% and higher one at 95.3%. This means that under the design conditions the dike often suffers from damages.

The failure probability of the existing dike due to overtopping is 47% for riprap-dike and 63 % for placed blocks-dike. This high uncertainty of safety of the dikes is due to the fact that the level of the dike's crest is too insufficient to the sea boundary. As stated in *Section 5.2.1,* when applying the latest Vietnamese code, the crest level of the dike corresponds to boundary conditions at design situation is 6.2 meters, and, 6.6 meter at present situation while it is 5.5 meters of the existing dikes. This failure probability is unaccepted by both Vietnamese and Dutch regulation. Therefore the dike's crest level needs to be enhanced.

Similarly, for failure mode of armour layers, the failure probability is 50% this implies that the chance which the dike is failure and not failure are equal. This can be considered that the working state of the dike reaches at critical state with the deterministic safe factor of 1.0. This is, generally, unaccepted by any design regulation. Therefore the size of elements of slope protection needs to be increased.

It is clear that the failures of the existing dikes mostly causes by overtopping and/or instability of armour layer of slope protection. The results of probabilistic calculation agree well with safety investigation of the dikes under deterministic approach in chapter 5. Moreover with higher boundary of overall probability of failure is 95.3 % it can be said that every storm season the dikes may suffer too much from damages. Reasonably, this exacts that what is happening at Namdinh dikes (see Problem Definition, Chapter 1 and Current Situation of Namdinh dikes, Chapter 2). The probabilities of failure of the dikes due to slopes sliding and piping modes are relatively small comparing to others.

The dikes with the required design parameters, which applied deterministic method in chapter 5, have failure probability in order of 4.5% to 6.4 % when using Vietnamese design code while it is from 2.8% to 5.5 % failure probability when using Dutch design code.

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

7.1.1 Conclusions on safety of the sea dikes in Namdinh

Sea dike in Namdinh have been a prevalent coastal defence, which provide protection of coastal areas from sea water flood and waves attacks. This system was established and has been developed over the century ago. For the time being, Namdinh sea dikes have suffered too much from damages with a large number of dikes breaches and reconstructions.

Based on analyse of historical record and development of Namdinh sea dikes the main reasons for dike failures and breaches were as follows:

1- Heavy foreshore erosion led to lowering the sand beach and foreland, formed scour holes in front of the dikes. As the consequences the toe of the dikes could not be stable. The failures of the toe resulted in series of consecutive damages of upper components of the dikes.

2- Increase in water depth in front of the dikes, as consequence of foreshore erosion, caused higher waves height (compared to the design wave height at the design situation) attacking directly on dike outer slope. On the other hand, the outer slope of most old dikes was not protected. That could lead to many damages of the outer slope.

3- During storm, high water level often accompanied with a surge and wave run-up caused too much overtopped water. This led to many damages of the dike components such as inner slope, outer slope and dike crest.

The study investigated all possible failure mechanisms of existing Namdinh sea dikes. The investigation carried out that failures of the dike system are various. These can be arranged in three aspects in terms of hydraulic, geotechnical and structural related failure mechanisms.

According to the deterministic assessment of the safety in this study, although there have been some dike upgrades and new construction in the recent years, the situation of Namdinh sea dikes is still at very low safety level. Wave overtopping, instability of outer slopes protections and instability of toe protections are the most critical and dominant failure modes.

The main causes of occurrences of these dike's failures are changes in boundary condition due to foreshore erosion and insufficient of geometrical design dimensions. It can be explained more as bellows:

- The existing dike's crest level is usually 1 meter (1 meter is relative to 20% of dike height) lower than that according to wave run-up criteria. This causes too much overtopping during high water level accompanying with a surge and leads to many damages of dikes.

- Applying actual design codes for investigation of stability of slope protection the study found that the rock (or block) size of existing slope protection (armour layer) is just ensured around 60% of the required size by Vietnamese code, about 55% by

Dutch code. Similarly, the rock size of toe protection is reached at 65% of the required rock size by applying both design codes.

- The total required thickness of armour layers, for given boundary condition of Namdinh sea dikes, based on the reshaped-equilibrium profile method is 0.85 meter, and, based on aspect of soil-mechanical stability is 0.8 meter while it is 0.5 meter for existing Namdinh revetments.
- Moreover, the boundary conditions in front of the dikes change as the result of foreshore erosion. Consequently, the water depth in front of dike increases. That leads to increase in the hydraulic loading (wave height) compared to the original (at design time) situation.

The investigation of design wave height has found that, since the depth limited wave height is applied (wave height is functioned of water depth, Hs = (0.45÷0.55)*Depth), the increase in design wave height between the original situation and present situation is about 20%.

The study on scour holes in front of the dike by applying various methods show that the maximum scour depth is about 1.2 meter. It is approximately 30% water depth of original situation. This means that the maximum relative increase of water depth will be about 30% possibly. Therefore the design wave height could be increase 30% when the equilibrium situation is reached.

Based on the results of analyses of geotechnical problems, including: calculation of seepage, analyses of stress-strain and displacements, analysis of slope stability, and piping calculation, it can be concluded that:

- Level of phreatic line is relatively so high that the seepage flows out at inner slope of the dike during high water level at the sea side. This is caused by not sufficient low permeability of dike's materials and insufficient thickness of clay layer under the outer slope. Consequently, the stability of the dike is decreased. Furthermore the seepage flows out at lower part of inner slope may take away soil particles and cause local erosion.
- The maximum settlement is 1.25% relatively to dike's height. This is accepted by actual design codes for soil structures.
- Outer and inner slopes are ensured stability conditions with the safety factor of 1.20 based on Bishop's method. The overall safety factor of the whole dike body is carried out by using PLAXIS. According to that, under the design condition, stability of the whole dike body can be ensured with safety factor of 1.10.
- Piping may not occur at Namdinh sea dikes as the result of piping calculation.

The probability of failures of existing Namdinh sea dikes are investigated by probabilistic calculations. The overall probability of dike failure is very high at 95.3%. This means that under the design conditions the dike will suffers too much from damages. Furthermore, according to probabilistic assessment, the failures of the existing dikes are mostly caused by overtopping and/or instability of armour layer of slope protection. The possibility at which the overtopping failure mode occurs is 64%. The probability of failure of armour layers is about 48%. This result is a good agreement with the deterministic assessment as well as prototype observations at Namdinh sea dikes.

According to probabilistic assessment, the failures of the existing dikes are mostly caused by overtopping and/or instability of armour layer of slope protection. These failure modes are considered the most critical and dominant mode of failure at Namdinh dikes. The possibility at which the overtopping failure mode occurs is 64%. The probability of failure of armour layers is 50%. These are unaccepted by any design standard.

The probabilistic sensitive analyses, of which stochastic variables contribute to the failure mechanism, state that the design water level (includes MHWL, surge) and design wave height give the most contribution to the probability of failures of these critical failure modes.

7.1.2 Conclusions on design of sea dikes in Vietnam

Most of the existing design of sea dikes and revetments in Vietnam were based on the old design methods (standards), which no longer be used nowadays, and were constructed under poor conditions of execution, quality control, investment and management. As the consequences the failures of sea dike system often occur during storms and typhoons.

To improve the design tools the new (actual) Vietnamese design code for sea dikes and revetments was published. In this design code, some updated design formulae were introduced and some advanced remarks were included. However there are still some limitations of using the code which can be summarised as follows:

- For wave run-up calculation: The formula referred from Chinese design code, GB50286-98, was introduced. When introducing the formula it is not clear how to determine related parameters (variables) in the formula.
- Determination of design crest level of sea dikes does not include relative sea level rise, expected settlement and wave overtopping criteria. This lead to underestimation of dike's crest level. For case study of Namdinh sea dikes the underestimation is in order of about 5% relatively to dike's height.
- For calculation of stability of armour layer: The formulae are not applied in proper form and coefficients. There is also lack of indications on the validity range of the formulae are and for which situations (see section 5.2.2.3).

Usually the actual design of sea dikes in Vietnam based on only present situation and boundary condition. It does not take in to account the changes of boundary conditions in the future (during serviced-life time) such as increase in water depth due to foreshore erosion and occurrence of scour holes, relative sea level rise, expected settlement etc. This may leads into underestimation of design parameters.

For present Namdinh boundary conditions, new dike design parameters are estimated by applying actual Vietnamese Code and Dutch Code. The result shows that the Dutch Code provides around 15% lager required dimensions.

7.2. Recommendations

It would be a good effort of this study to investigate the safety of all Namdinh sea dike sections based on the full analyse of failure mechanisms, their causes and probabilities of occurrence. However, due to the limitation of time, data and information the study does not go deeply into all aspects of the whole system. The analyse is just carried out for one representative section of the dike system in terms of investigation of hydraulic boundaries and some main hydraulic, geotechnical and structural related failure mechanisms.

Nevertheless, the study allows formulating the following recommendations:

For the Namdinh sea dikes, which are threatened by frequent failures and designed at very low safety level, improvement and reinforcement of the dike's elements are necessary. The level of dike crest should be sufficient high to avoid the damage due to wave overtopping. The dike's slope and the toe should be protected by larger size of elements. The toe should be often protected to a larger extent. In order to avoid the possible failure due to seepage flow, the thickness of clay layer under outer slope should be increase and the gravel bund (or drainage tool) should be provided at lower part of inner slope.

The applied cross section of upgraded and new dikes should be based on optimal design which takes in to account also re-used material of the exiting dikes. Providing the berm at outer slope is suggested.

At some places such as HaiTrieu, Vanly and Haily, where the heavy erosion takes place, using dikes in combination with other coastal defensive measure should be considered. The studies and/or pilot projects on the effectiveness of the combined measures should be carried out before decision is made for the whole system.

Safety assessment of the dikes system in Namdinh should be carried out for all sections. Probabilistic assessment and risk analysis should be performed for the whole system. In order to be able to do such analysis in a proper way, much more data need to be collected and should be established a good monitoring and management system/strategy for the dike system. This could be the subject for further study.

For the new design of sea dikes and revetments in Vietnam the latest Vietnamese design code could be a reference. However, it is necessary to compare the outcome with more actual design codes (e.g. Dutch code, British code, U.S code…) in Vietnamese condition.

When applying the latest Vietnamese design code of sea dikes the following aspects need to be considered, from the engineering point of view:

- Determination of load and strength boundary conditions. Especially the estimation of hydraulic boundary condition such as the design water level, design wave height and wave period etc. The depth limited wave height could be a good reference for any method of design wave height estimation
- Be aware clearly of the limitation of formulae when applying for the design. The validation range of applying the formula should be known. The definition of the variables in the formulae must be well understood.

- The applied formulae in present design code for wave run up and stability of slope protection should be compared with more actual formulae such as defined by Van der Meer.
- The criteria of wave overtopping, the value of sea level rise and expected settlement should (must) be added when determine the design crest level.
- Calculation of required size of rock for toe protection should not only account for impact of waves but also the effect of currents on stability of applied material.
- Considerations of operation and maintenance should be included during design process.

- The probabilistic design and risk analysis should be applied for the new design of sea dikes as well as coastal structures.
- Proper supervision and quality control during execution is a must.
- Periodic safety assessment (monitoring of the actual state of the dikes/coastal structures) should be implemented.

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APPENDICES

SUB-TABLE OF CONTENTS

APPENDIX 1 USING PLAXIS FOR CALCULATION OF GEOTECHNICAL PROBLEMS.

PLAXIS was developed by first by Technical University of Delft in 1987 and then continuously grown by PLAXIS B.V Company since 1993. This is a special purpose two-dimensional finite element computer program used to perform deformation and stability analyses for various types of geotechnical applications. Real situations may be modelled either by a plane strain or an axisymmetric model.

In this appendix some scientific background is given of the theories and numerical methods on which the PLAXIS program is based on. The appendix contains the summary on groundwater flow theory, deformation theory and consolidation theory. In the next part of the appendix a global calculation scheme is provided for a plastic deformation analysis. Following by, the calculation results will be presented under the form of geometry plot of the dike cross section.

1. Groundwater flow theory.

In this appendix the theory of groundwater flow will be review as used in PLAxIS. In addition to a general description of groundwater flow, attention is focused on the finite element formulation.

1.1 Basic equation of steady flow

Flow in a porous medium can be described by Darcy's law. Considering flow in a vertical x-y-plane the following equations apply:

$$q_x = -k_x \frac{\partial \phi}{\partial x} \qquad q_y = -k_y \frac{\partial \phi}{\partial y} \tag{1-1}$$

The equations express that the specific discharge, q, follows from the permeability, k, and the gradient of the groundwater head. The head, ϕ, is defined as follows:

$$\phi = y - \frac{p}{\gamma_w} \tag{1.2}$$

where y is the vertical position, p is the stress in the pore fluid (negative for pressure) and γ_w is the unit weight of the pore fluid. For steady flow the continuity condition applies:

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = 0 \tag{1.3}$$

Eq. (1.3) expresses that there is no net inflow or outflow in an elementary area, as illustrated in Figure 1.

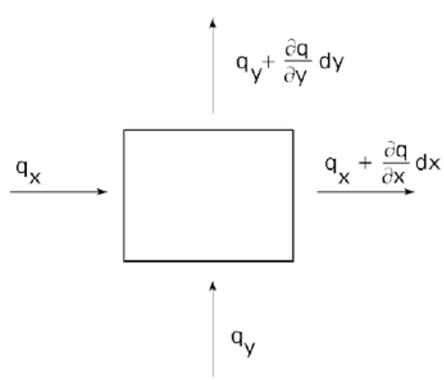


Figure 1: Illustration of continuity condition.

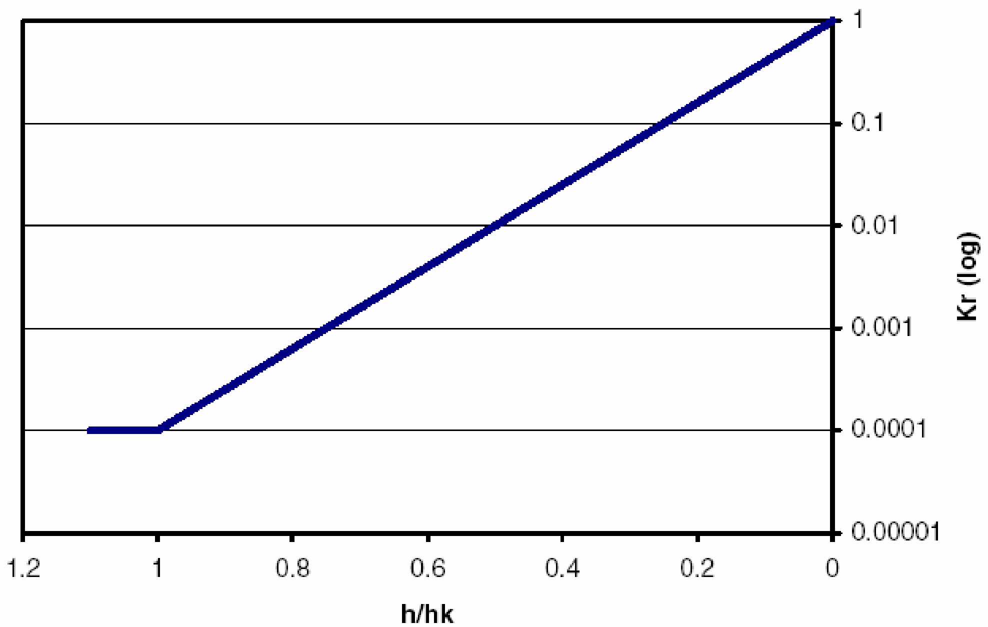
1.2 Finite element discretisation.

The groundwater head in any position within an element can be expressed in the values at the nodes of that element:

$$\phi(\xi,\eta) = \underline{N}\,\underline{\phi}^e \qquad (1.4)$$

where $\underline{N}$ is the vector with interpolation functions and ξ and η are the local coordinates within the element. According to Eq. (1.1) the specific discharge is based on the gradient of the groundwater head. This gradient can be determined by means of the $\underline{B}$-matrix, which contains the spatial derivatives of the interpolation functions. In order to describe flow for saturated soil (underneath the phreatic line) as well as non-saturated soil (above the phreatic line), a reduction function K^r is introduced in Darcy's law (Desai, 1976; Li & Desai, 1983; Bakker, 1989):

$$q_x = -K^r k_x \frac{\partial \phi}{\partial x} \qquad q_y = -K^r k_y \frac{\partial \phi}{\partial y} \qquad (1.5)$$



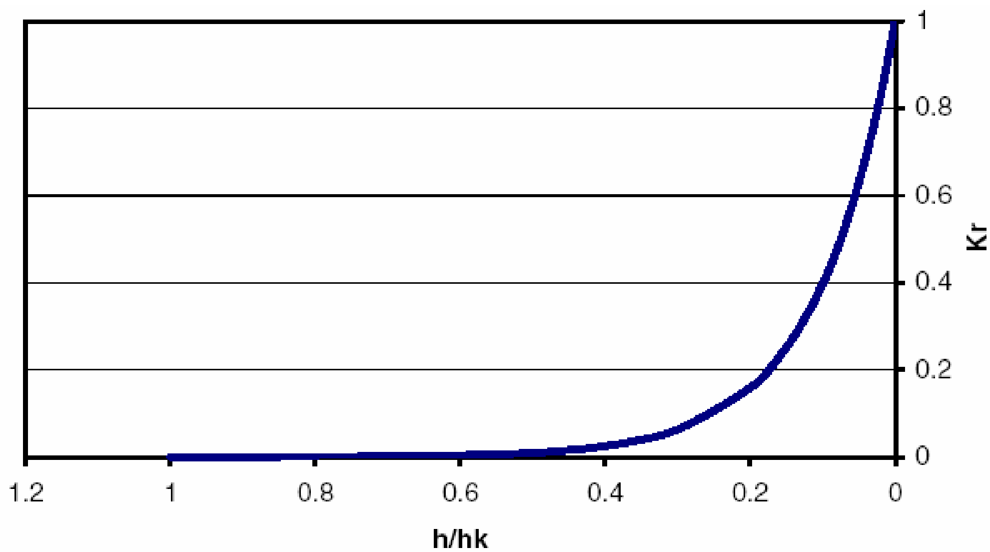


Figure 2: Adjustment of the permeability between saturated (a) and unsaturated (b) zones (K' = ratio of permeability over saturated permeability)

The reduction function has a value of 1 below the phreatic line (compressive pore pressures) and has lower values above the phreatic line (tensile pore pressures). In the transition zone above the phreatic line, the function value decreases to the minimum of 10^{-4}.
In the transition zone the function is described using a log-linear relation:

$K^r = 10^{-4h/h_k}$ $\qquad$ $10^{-4} \leq K^r \leq 1$
Or

$$^{10}\log(K^r) = -\frac{4h}{h_k} \qquad (1\text{-}6)$$

where h is the pressure head and h_k is the pressure head where the reduction function has reached the minimum of 10^{-4} . In PLAXIS h_k has a default value of 0.7 m (independent of the chosen length unit).

In the numerical formulation, the specific discharge, $\underline{q}$, is written as:

$$\underline{q} = -K^r \underline{\underline{R}}\,\underline{\underline{B}}\phi^e \qquad (1\text{-}7)$$

where

$$\underline{q} = \begin{bmatrix} q_x \\ q_y \end{bmatrix} \qquad \text{and:} \qquad \underline{\underline{R}} = \begin{bmatrix} k_x & 0 \\ 0 & k_y \end{bmatrix} \qquad (1\text{-}8)$$

From the specific discharges in the integration points, $\underline{q}$, the nodal discharges Q^e can be integrated according to:

$$Q^e = - \int \underline{\underline{B}}^T \; \underline{q} \; dV \qquad (1\text{-}9)$$

in which $\underline{B}^T$ is the transpose of the B-matrix. On the element level the following equations apply:

$$Q^e = \underline{\underline{K}}^e \underline{\phi}^e \quad \text{With :} \quad \underline{\underline{K}}^e = \int K^r \, \underline{\underline{B}}^T \; \underline{\underline{R}} \; \underline{\underline{B}} \; dV \qquad (1\text{-}10)$$

On a global level, contributions of all elements are added and boundary conditions (either on the groundwater head or on the discharge) are imposed. This results in a set of n equations with n unknowns:

$$\boldsymbol{Q = K . \phi} \qquad (1.11)$$

in which $\underline{K}$ is the global flow matrix and Q contains the prescribed discharges that are given by the boundary conditions.

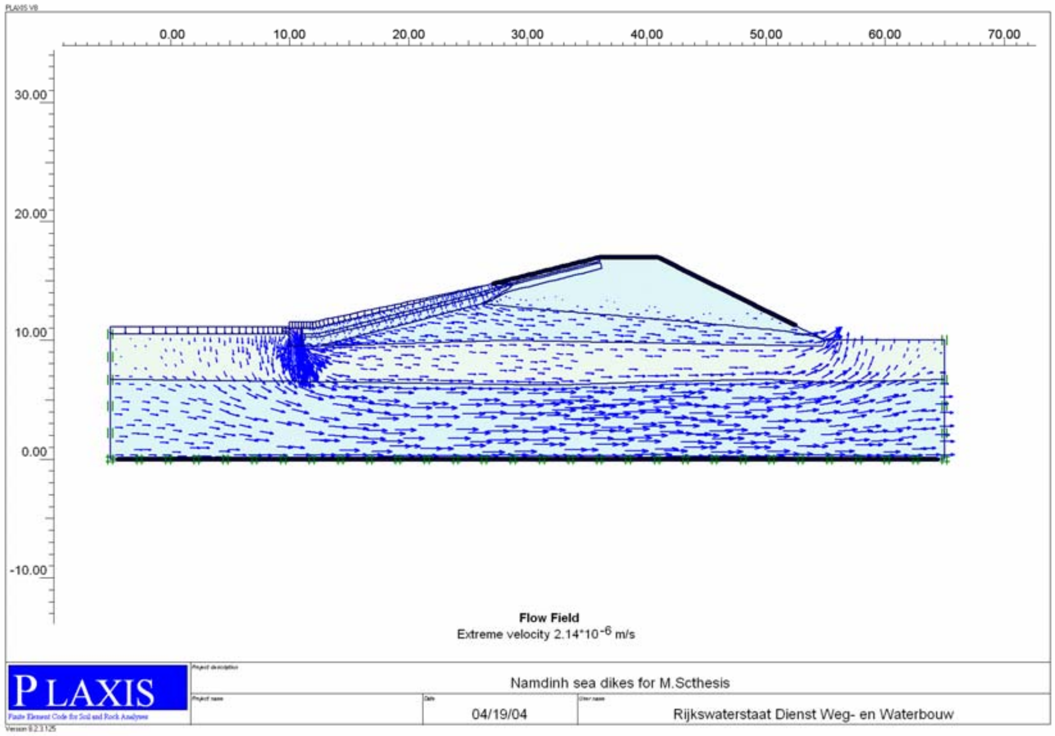
In the case that the phreatic line is unknown (unconfined problems), a Picard scheme is used to solve the system of equations iteratively. The linear set is solved in incremental form and the iteration process can be formulated as:

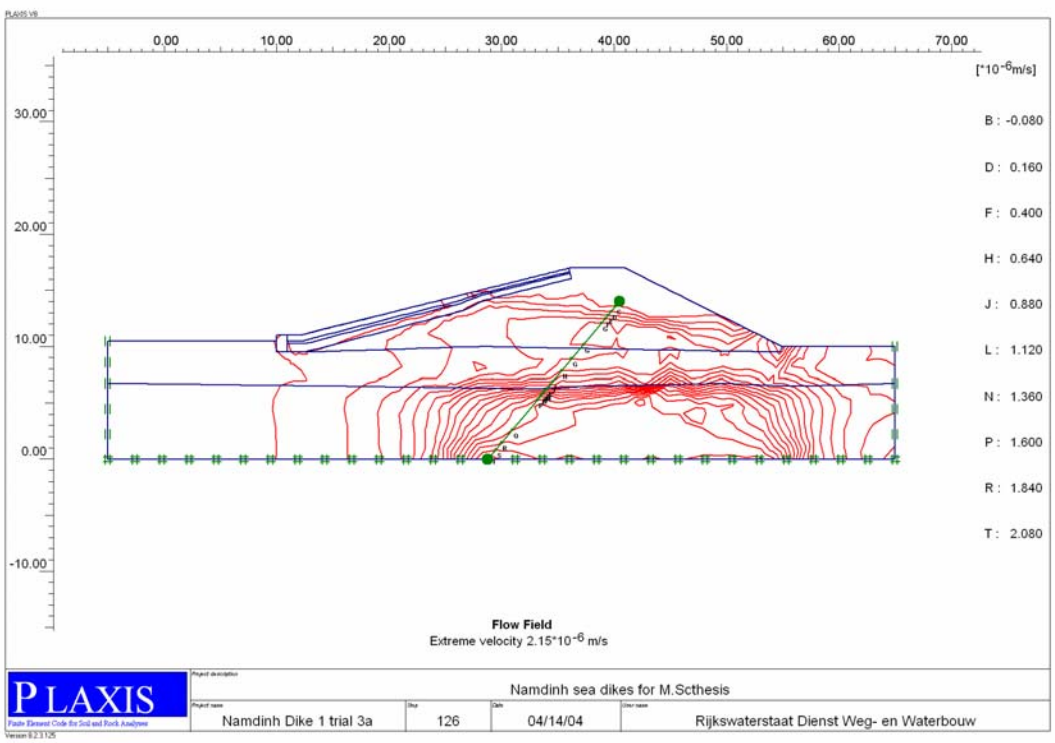
$$K^{j-1}\delta\underline{\phi}^j = Q - K^{j-1}\phi^{j-1} \qquad \underline{\phi}^j = \phi^{j-1} + \delta\phi^j \qquad (1.12)$$

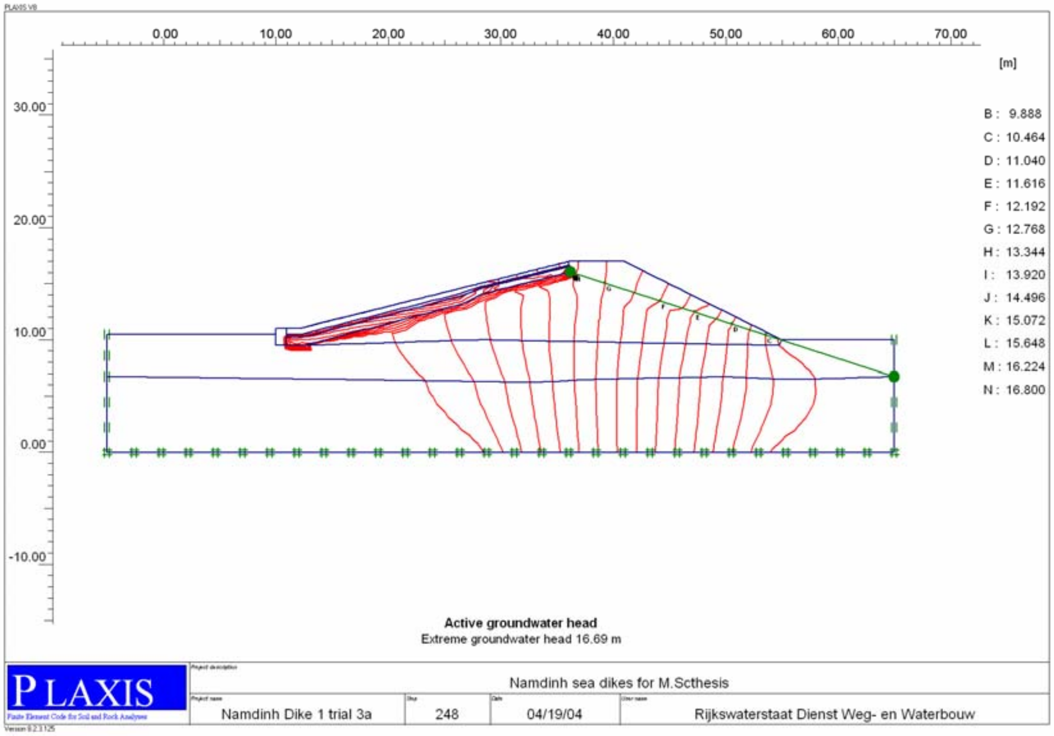
in which j is the iteration number and $\underline{r}$ is the unbalance vector. In each iteration increments of the groundwater head are calculated from the unbalance in the nodal discharges and added to the active head.

From the new distribution of the groundwater head the new specific discharges are calculated according to Eq. (1.7), which can again be integrated into nodal discharges. This process is continued until the norm of the unbalance vector, i.e. the error in the nodal discharges, is smaller than the tolerated error.

1.4 Result from calculation of seepage.







[m]
B : 9.888
C : 10.464
D : 11.040
E : 11.616
F : 12.192
G : 12.768
H : 13.344
I : 13.920
J : 14.496
K : 15.072
L : 15.648
M : 16.224
N : 16.800
Active groundwater head
Extreme groundwater head 16.69 m
PLAXIS
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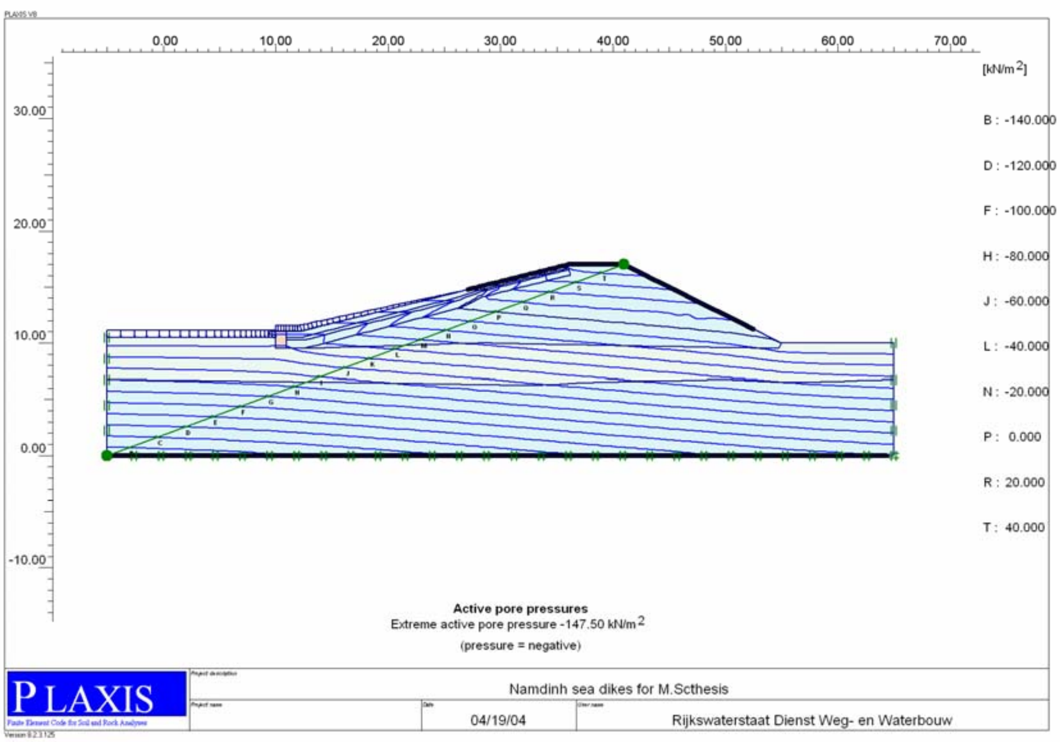
Rijkswaterstaat Dienst Weg- en Waterbouw



[kN/m²]
B : -140.000
D : -120.000
F : -100.000
H : -80.000
J : -60.000
L : -40.000
N : -20.000
P : 0.000
R : 20.000
T : 40.000
Active pore pressures
Extreme active pore pressure -147.50 kN/m²
(pressure = negative)
PLAXIS
Namdinh sea dikes for M.Scthesis
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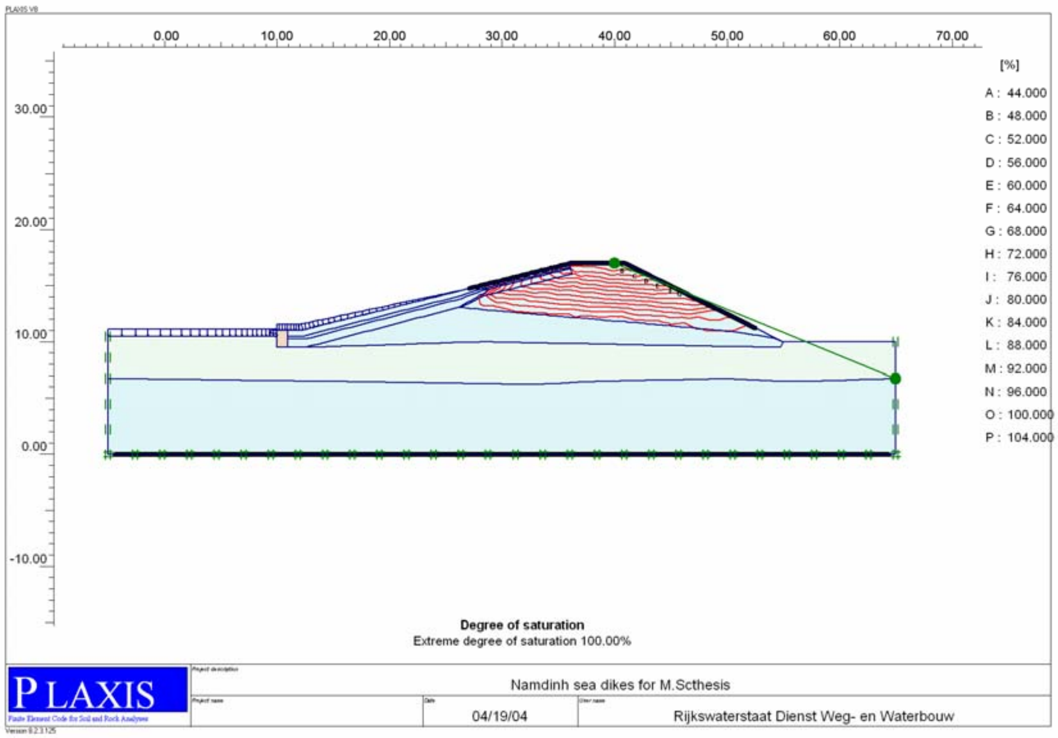
Rijkswaterstaat Dienst Weg- en Waterbouw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Degree of saturation
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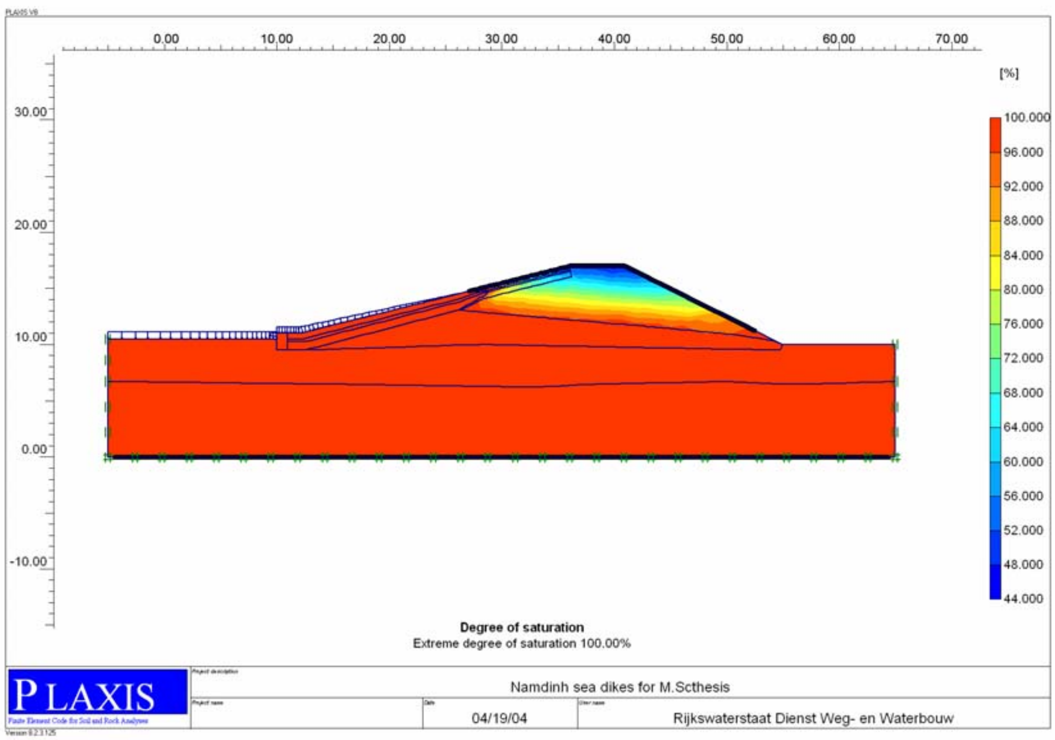
Rijkswaterstaat Dienst Weg- en Waterbouw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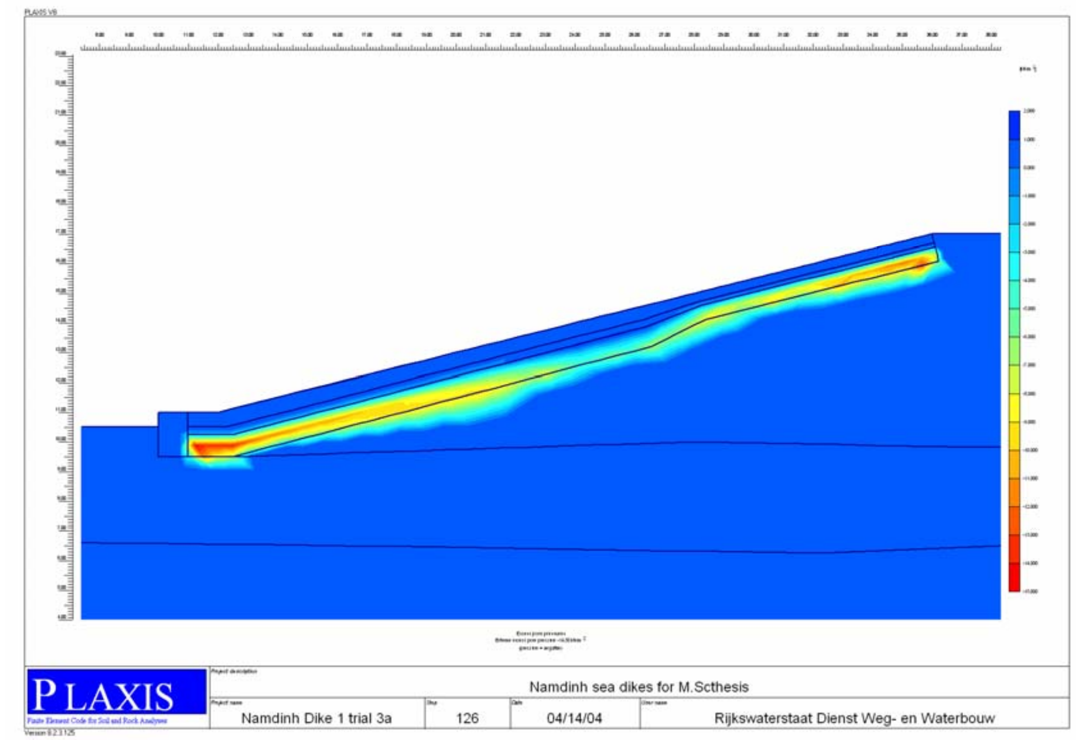
PLAXIS
Namdinh sea dikes for M.Scthesis
04/19/04
Rijkswaterstaat Dienst Weg- en Waterbouw



2. Deformation and consolidation theory

The basic equations for the static deformation of a soil body are formulated within the framework of continuum mechanics. A restriction is made in the sense that deformations are considered to be small. This enables a formulation with reference to the original undeformed geometry. The continuum description is discretised according to the finite element method.

2.1 Basic equations of continuum deformation.

The static equilibrium of a continuum can be formulated as:

$$\underline{\underline{L}}^T \underline{\sigma} + \underline{p} = \underline{0} \qquad (2.1)$$

This equation relates the spatial derivatives of the six stress components, assembled in vector $\underline{\sigma}$, to the three components of the body forces, assembled in vector $\underline{p}.\underline{\underline{L}}^T$ is the transpose of a differential operator, defined as:

$$\underline{\underline{L}}^T = \begin{bmatrix} \frac{\partial}{\partial x} & 0 & 0 & \frac{\partial}{\partial y} & 0 & \frac{\partial}{\partial z} \\ 0 & \frac{\partial}{\partial y} & 0 & \frac{\partial}{\partial x} & \frac{\partial}{\partial z} & 0 \\ 0 & 0 & \frac{\partial}{\partial z} & 0 & \frac{\partial}{\partial y} & \frac{\partial}{\partial x} \end{bmatrix} \qquad (2.2)$$

In addition to the equilibrium equation, the kinematic relation can be formulated as:

$$\underline{\varepsilon} = \underline{\underline{L}}^T . \underline{u} \qquad (2.3)$$

This equation expresses the six strain components, assembled in vector $\underline{\varepsilon}$, as the spatial derivatives of the three displacement components, assembled in vector $\underline{u}$, using the previously defined differential operator $\underline{\underline{L}}$. The link between Eq. (2.1) and (2.3) is formed by a constitutive relation representing the material behavior. The general relation between rates of stress and strain is:

$$\underline{\dot{\sigma}} = \underline{\underline{M}}\ \underline{\dot{\varepsilon}} \qquad (2.4)$$

The combination of Eqs. (2.1), (2.3) and (2.4) would lead to a second-order partial differential equation in the displacements $\underline{u}$.

However, instead of a direct combination, the equilibrium equation is reformulated in a weak form according to Galerkin's variation principle (see among others Ziefikiewicz, 1967):

$$\int \delta\, \underline{u}^T \left(\underline{\underline{L}}^T \underline{\sigma} + \underline{p} \right) dV = 0 \qquad (2.5)$$

In this formulation 6 u represents a kinematically admissible variation of displacements. Applying Green's theorem for partial integration to the first term in Eq. (2.5) leads to:

$$\int \delta\, \underline{\varepsilon}^T \underline{\sigma}\, dV = \int \delta\, \underline{u}^T\, \underline{p}\, dV + \int \delta\, \underline{u}^T\, \underline{t}\, dS \qquad (2.6)$$

This introduces a boundary integral in which the boundary traction appears. The three components of the boundary traction are assembled in the vector t. Eq. (2.6) is referred to as the virtual work equation.

The development of the stress state can be regarded as an incremental process:

$$\underline{\sigma}^i = \underline{\sigma}^{i\text{-}1} + \Delta\underline{\sigma} \qquad \Delta\underline{\sigma} = \int \underline{\dot{\sigma}}\, d\,t \qquad (2.7)$$

In this relation σ^i represents the actual state of stress which is unknown and σ^{i-1}represents the previous state of stress which is known. The stress increment is the stress rate integrated over a small time increment.

If Eq. (2.6) is considered for the actual state i, the unknown stresses σ^i can be eliminated using Eq. (2.7):

$$\int \delta\, \underline{\varepsilon}^T\ \Delta\underline{\sigma}\, dV = \int \delta\, \underline{u}^T\, \underline{p}^i\ dV + \int \delta\, \underline{u}^T\, \underline{t}^{\,i}\, dS - \int \delta\, \underline{\varepsilon}^T\, \underline{\sigma}^{i\text{-}1}\, dV \qquad (2.8)$$

It should be noted that all quantities appearing in Eqs. (2. 1) to (2.8) are functions of the position in the three-dimensional space.

2.2 Finite element discretisation

According to the finite element method a continuum is divided into a number of (volume) elements. Each element consists of a number of nodes. Each node has a number of degrees of freedom that correspond to discrete values of the unknowns in the boundary value problem to be solved. In the present case of deformation theory the degrees of freedom correspond to the displacement components. Within an element the displacement field u is obtained from the discrete nodal values in a vector v using interpolation functions assembled in matrix N:

$$\underline{u} = \underline{\underline{N}}\, \underline{v} \qquad (2.9)$$

The interpolation functions in matrix N are often denoted as shape ftmctions. Substitution of Eq. (2.9) in the kinematic relation (2.3) gives:

$$\underline{\varepsilon} = \underline{\underline{L}}\,\underline{\underline{N}}\,\underline{v} = \underline{\underline{B}}\,\underline{v} \tag{2.10}$$

In this relation B is the strain interpolation matrix, which contains the spatial derivatives of the interpolation ftmctions. Eqs. (2.9) and (2. 10) can be used in variational, incremental and rate form as well. Eq. (2.8) can now be reformulated in discretised form as:

$$\int \left(\underline{\underline{B}}\,\delta \underline{v}\right)^T \Delta\underline{\sigma}\, dV = \int \left(\underline{\underline{N}}\,\delta \underline{v}\right)^T \underline{p}^i\, dV + \int \left(\underline{\underline{N}}\,\delta \underline{v}\right)^T \underline{t}^i\, dS - \int \left(\underline{\underline{B}}\,\delta \underline{v}\right)^T \underline{\sigma}^{i\text{-}1}\, dV \tag{2.11}$$

The discrete displacements can be placed outside the integral:

$$\delta\,\underline{v}^T \int \underline{\underline{B}}^T \Delta\underline{\sigma} dV = \delta \underline{v}^T \int \underline{\underline{N}}^T\, \underline{p}^i\, dV + \delta \underline{v}^T \int \underline{\underline{N}}^T \underline{t}^i\; dS - \delta \underline{v}^T \int \underline{\underline{B}}^T \underline{\sigma}^{i\text{-}1} dV \tag{2.12}$$

Provided that Eq. (2.12) holds for any kinematically admissible displacement variation, the equation can be written as:

$$\int \underline{\underline{B}}^T\, \Delta\underline{\sigma}\;\; d\,V = \int \underline{\underline{N}}^T\;\, \underline{p}^i\, d\,V + \int \underline{\underline{N}}^T\, \underline{t}^{\,i}\; d\;S - \int \underline{\underline{B}}^T\;\, \underline{\sigma}^{i\text{-}1}\, d\,V \tag{2.13}$$

The above equation is the elaborated equilibrium condition in discretised form. The first term on the right-hand side together with the second term represent the current external force vector and the last term represents the internal reaction vector from the previous step. A difference between the external force vector and the internal reaction vector should be balanced by a stress increment.

The relation between stress increments and strain increments is usually non-linear. As a result, strain increments can generally not be calculated directly, and global iterative procedures are required to satisfy the equilibrium condition (2.13) for all material points. Global iterative procedures are described later in Section 2.5, but the attention is first focused on the (local) integration of stresses.

2.3 Implicit integration of differential plasticity model.

The stress increments are obtained by integration of the stress rates according to Eq. (2.7). For differential plasticity models the stress increments can generally be written as:

$$\Delta\underline{\sigma} = \underline{\underline{D}}^e \left(\Delta\underline{\varepsilon} - \Delta\,\underline{\varepsilon}^p\right) \tag{2.14}$$

In this relation D e represents the elastic material matrix for the current stress increment. The strain increments AE are obtained from the displacement increments Av using the strain interpolation matrix B, similar to Eq. (2. 10).

For elastic material behaviour, the plastic strain increment Ae is zero. For plastic material behaviour, the plastic strain increment can be written, according to Vermeer (1979), as:

$$\Delta\,\underline{\varepsilon}^p = \Delta\lambda \left[\; (1-\omega)\left(\frac{\partial g}{\partial \underline{\sigma}}\right)^{i\text{-}1} + \omega\left(\frac{\partial g}{\partial \underline{\sigma}}\right)^{i}\;\right] \tag{2.15}$$

In this equation $\Delta\lambda$ is the increment of the plastic multiplier and ω is a parameter indicating the type of time integration. For $\omega = 0$ the integration is called explicit and for $\omega = 1$ the integration is called implicit. For $\omega = 1$ the equation (2.15) reduced to

$$\Delta \underline{\varepsilon}^{p} = \Delta\lambda \left(\frac{\partial g}{\partial \underline{\sigma}} \right)^{i} \tag{2.16}$$

Substitution of Eq. (2.16) into Eq. (2.14) and successively into Eq. (2.7) gives:

$$\underline{\sigma}^{i} = \underline{\sigma}^{tr} - \Delta\lambda \, \underline{\underline{D}}^{e} \left(\frac{\partial g}{\partial \underline{\sigma}} \right)^{i} \qquad \text{with:} \qquad \underline{\sigma}^{tr} = \underline{\sigma}^{i\text{-}1} + \underline{\underline{D}}^{e} \, \Delta\underline{\varepsilon} \tag{2.17}$$

In this relation σ^{i} is an auxiliary stress vector, referred to as the elastic stresses or trial stresses, which is the new stress state when considering purely linear elastic material behaviour.

The increment of the plastic multiplier AA, as used in Eq. (2.17), can be solved from the condition that the new stress state has to satisfy the yield condition:

$$f\left(\underline{\sigma}^{i}\right) = 0 \tag{2.18}$$

For perfectly-plastic and linear hardening models the increment of the plastic multiplier can be written as:

$$\Delta\lambda = \frac{f\left(\underline{\sigma}^{tr}\right)}{d + h} \tag{2.19}$$

$$d = \left(\frac{\partial f}{\partial \underline{\sigma}} \right)^{\underline{\sigma}^{tr}} \underline{\underline{D}}^{e} \left(\frac{\partial g}{\partial \underline{\sigma}} \right)^{i} \tag{2.20}$$

The symbol h denotes the hardening parameter, which is zero for perfectly-plastic models and constant for linear hardening models. In the latter case the new stress state can be formulated as:

$$\underline{\sigma}^{i} = \underline{\sigma}^{tr} - \frac{\left\langle f\left(\underline{\sigma}^{tr}\right) \right\rangle}{d + h} \underline{\underline{D}}^{e} \left(\frac{\partial g}{\partial \underline{\sigma}} \right)^{i} \tag{2.21}$$

The ( )-brackets are referred to as McCauley brackets, which have the following convention:

(x) = 0 for: $x \leq 0$ and: (x) = x for: $x > 0$

2.4 Global iterative procedure.

Substitution of the relationship between increments of stress and increments of strain, $\Delta\underline{\sigma} = \underline{\underline{M}} \, \Delta\underline{\varepsilon}$ into the equilibrium equation (2.13) leads to:

$$\underline{\underline{K}}^{i} \, \Delta \underline{v}^{i} = \underline{f}_{ex}^{\,i} - \underline{f}_{in}^{\,i\text{-}1} \tag{2.22}$$

In this equation K is a stiffness matrix, $\Delta\underline{v}$ is the incremental displacement vector, $\underline{f}_{ex}$ is the external force vector and $\underline{f}_{in}$, is the internal reaction vector. The superscript i refers to the step number. However, because the relation between stress increments and strain increments is generally non-linear, the stiffness matrix cannot be formulated exactly beforehand. Hence, a global iterative procedure is required to satisfy both the equilibrium condition and the constitutive relation. The global iteration process can be written as:

$$\underline{\underline{K}}^{j} \, \delta \underline{v}^{j} = \underline{f}_{ex}^{\,i} - \underline{f}_{in}^{\,j\text{-}1} \tag{2.23}$$

The superscript j refers to the iteration number. 6v is a vector containing sub-incremental displacements, which contribute to the displacement increments of step i:

$$\Delta \underline{v}^i = \sum_{j=1}^{n} \delta \underline{v}^j \qquad (2.24)$$

where n is the number of iterations within step i. The stiffness matrix K, as used in Eq. (2.23), represents the material behaviour in an approximated manner. The more accurate the stiffness matrix, the fewer iterations are required to obtain equilibrium within a certain tolerance.

In its simplest form K represents a linear-elastic response. In this case the stiffness matrix can be formulated as:

$$\underline{\underline{K}} = \int \underline{\underline{B}}^T \underline{\underline{D}}^e \underline{\underline{B}}\, dV \qquad \text{(elastic stiffness matrix)} \quad (2.25)$$

where D^e is the elastic material matrix according to Hooke's law and $\underline{B}$ is the strain interpolation matrix. The use of an elastic stiffness matrix gives a robust iterative procedure as long as the material stiffness does not increase, even when using non-associated plasticity models. Special techniques such as arc-length control (Riks, 1979), over-relaxation and extrapolation (Vermeer & Van Langen, 1989) can be used to improve the iteration process. Moreover, the automatic step size procedure, as introduced by Van Langen & Vermeer (1990), can be used to improve the practical applicability. For material models with linear behaviour in the elastic domain, such as the standard Mohr-Coulomb model, the use of an elastic stiffness matrix is particularly favourable, as the stiffness matrix needs only be formed and decomposed before the first calculation step. This calculation procedure is summarised in section 2.6.

2.5 Basic equation of consolidation theory.

The governing equations of consolidation as used in PLAXIS follow Biot's theory (Biot, 1956). Darcy's law for fluid flow and elastic behaviour of the soil skeleton are also assumed. The formulation is based on small strain theory. According to Terzaghi's principle, stresses are divided into effective stresses and pore pressures:

$$\underline{\sigma} = \underline{\sigma}' + \underline{m}\left(p_{steady} + p_{excess}\right) \qquad (3.1)$$

where: $\underline{\sigma} = (\sigma_{xx} \sigma_{yy} \sigma_{zz} \sigma_{xy} \sigma_{yz} \sigma_{zx})^T$ and $\underline{m}=(1\ 1\ 1\ 0\ 0\ 0)^T$ (3.2)

a is the vector with total stresses, d contains the effective stresses~ pexes is the excess pore pressure and m is a vector containing unity terms for normal stress components and zero terms for the shear stress components. The steady state solution at the end of the consolidation process is denoted as p_{steady}. Within PLAXIS p_{steady} is defined as:

$$p_{steady} = \sum Mweight \cdot p_{input} \qquad (3.3)$$

where p_{input} is the pore pressure generated in the input program based phreatic lines or on a groundwater flow calculation. Note that within PLAXIS compressive stresses are considered to be negative; this applies to effective stresses as well as to pore pressures. In fact it would be more appropriate to refer to p_{input} and p_{steady} as pore stresses, rather than pressures. However, the term pore pressure is retained, although it is positive for tension.

The constitutive equation is written in incremental form. Denoting an effective stress increment as d' and a strain increment as the constitutive equation is:

$$\dot{\underline{\sigma}}' = \underline{\underline{M}}\, \dot{\underline{\varepsilon}} \qquad (3.4)$$

where: $\underline{\varepsilon}=(\varepsilon_{xx}\ \varepsilon_{yy}\ \varepsilon_{zz}\ \gamma_{xy}\ \gamma_{yz}\ \gamma_{zx})^T$ (3.5)

and $\underline{M}$ represents the material stiffness matrix.

By applying similar method as in Deformation section (section 2.2), here the finite element approximation is used to discrete the continuous components and fully implicit scheme is used for the problem.

2.6 Calculation procedure [PLAXIS 8.00 User's guide]

Finite element calculation process based on the elastic stiffness matrix

Read input data

Form stiffness matrix $\underline{\underline{K}} = \int \underline{\underline{B}}^T \ \underline{\underline{D}}^e \ \underline{\underline{B}} \, d\,V$

New step $i \rightarrow i+1$

Form new load vector $\underline{f}^i_{ex} = \underline{f}^{i\text{-}1}_{ex} + \Delta \underline{f}_{ex}$

Form reaction vector $\underline{f}_{in} = \int \underline{\underline{B}}^T \underline{\sigma}^{i\text{-}1}_c \, d\,V$

Calculate unbalance $\Delta \underline{f} = \underline{f}^i_{ex} - \underline{f}_{in}$

Reset displacement increment $\Delta \underline{v} = 0$

New iteration $j \rightarrow j+1$

Solve displacements $\delta \underline{v} = \underline{\underline{K}}^{-1} \Delta \underline{f}$

Update displacement increments $\Delta \underline{v}^j = \Delta \underline{v}^{j\text{-}1} + \delta \underline{v}$

Calculate strain increments $\Delta \underline{\varepsilon} = \underline{\underline{B}} \Delta \underline{v}$; $\delta \underline{\varepsilon} = \underline{\underline{B}} \delta \underline{v}$

Calculate stresses: Elastic $\underline{\sigma}^{tr} = \underline{\sigma}^{i\text{-}1}_c + \underline{\underline{D}}^e \Delta \underline{\varepsilon}$

Equilibrium $\underline{\sigma}^{eq} = \underline{\sigma}^{i,j\text{-}1}_c + \underline{\underline{D}}^e \delta \underline{\varepsilon}$

Constitutive $\underline{\sigma}^{i,j}_c = \underline{\sigma}^{tr} - \dfrac{\langle f(\underline{\sigma}^{tr}) \rangle}{d} \underline{\underline{D}}^e \dfrac{\partial g}{\partial \underline{\sigma}}$

Form reaction vector $\underline{f}_{in} = \int \underline{\underline{B}}^T \underline{\sigma}^{i,j}_c \, d\,V$

Calculate unbalance $\Delta \underline{f} = \underline{f}^i_{ex} - \underline{f}_{in}$

Calculate error $e = \dfrac{|\Delta \underline{f}|}{|\underline{f}^i_{ex}|}$

Accuracy check if $e > e_{tolerated} \rightarrow$ new iteration

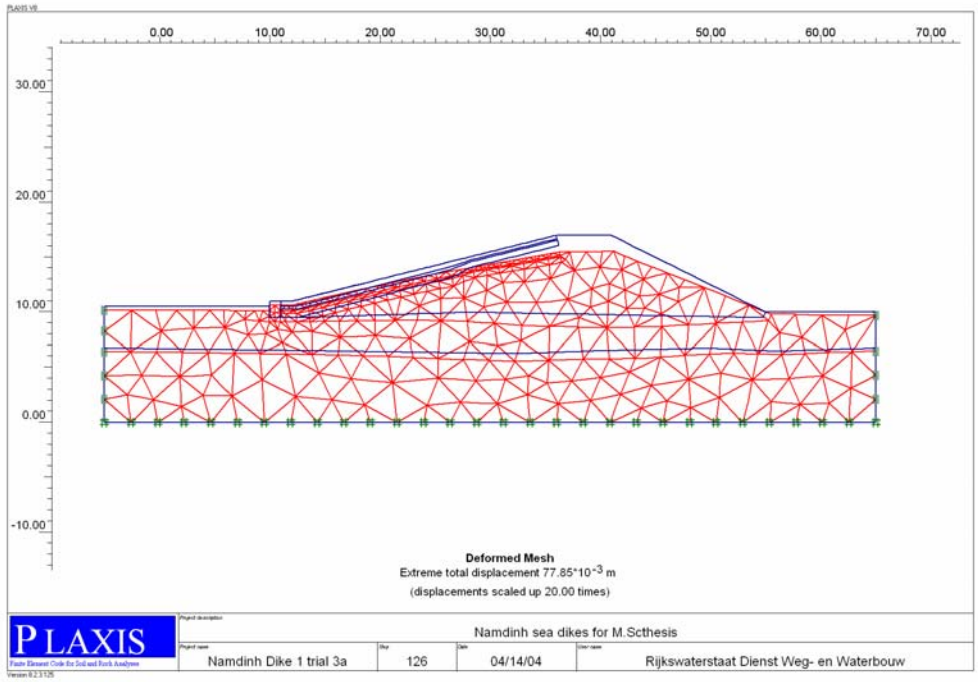
Update displacements $\underline{v}^i = \underline{v}^{i\text{-}1} + \Delta \underline{v}$

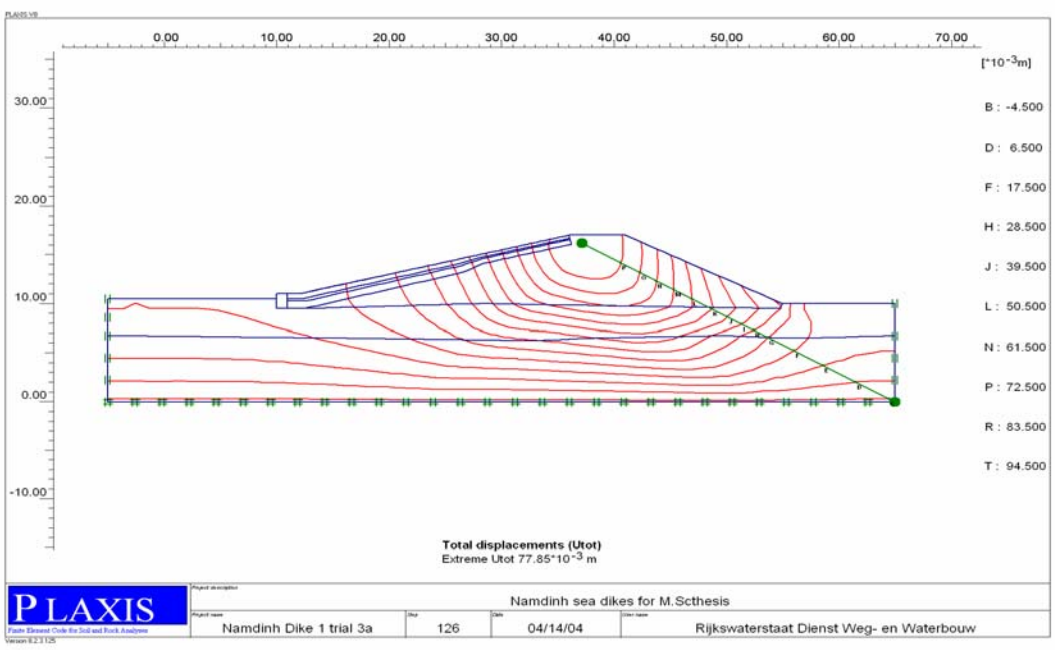
Write output data (results)

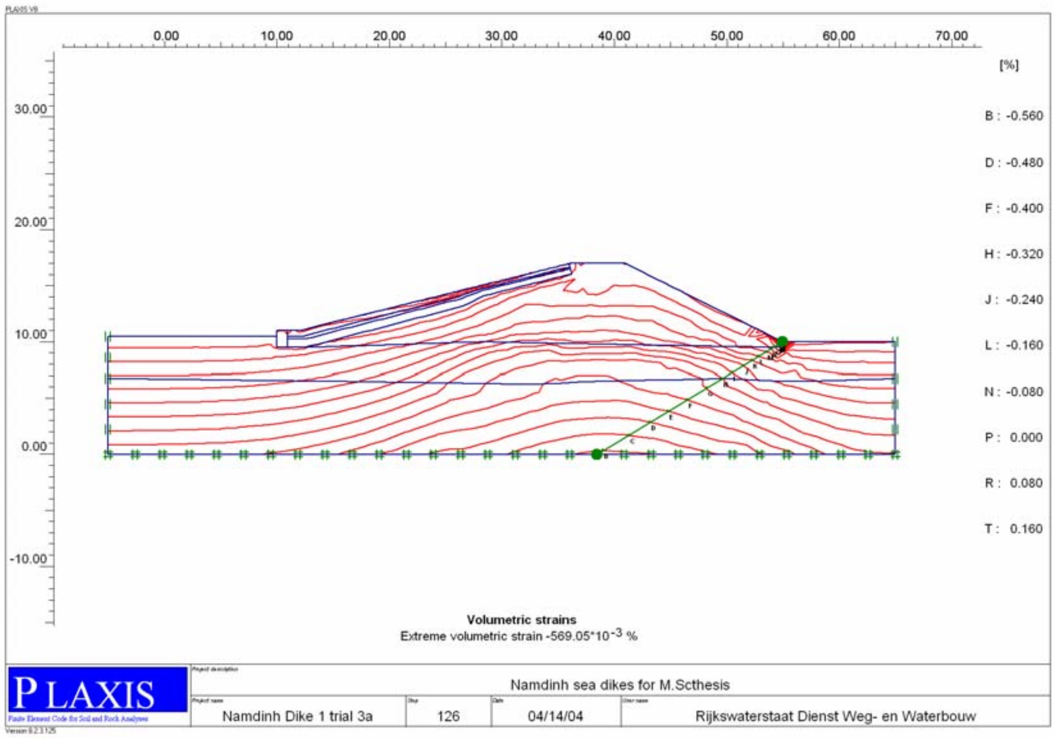
If not finished $\rightarrow$ new step

Finish

2.8 Result of calculation geotechnical problems.



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H : -0.320
J : -0.240
L : -0.160
N : -0.080
P : 0.000
R : 0.080
T : 0.160
Volumetric strains
Extreme volumetric strain -569.05*10^-3 %
PLAXIS
Namdinh sea dikes for M.Scthesis
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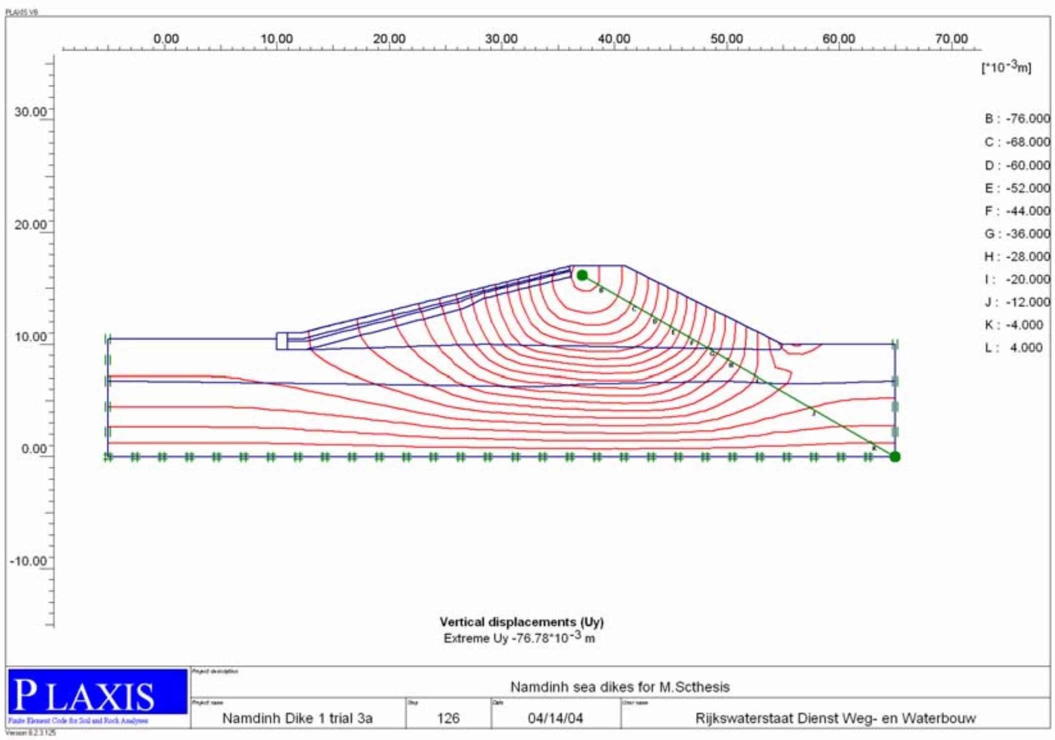
Rijkswaterstaat Dienst Weg- en Waterbouw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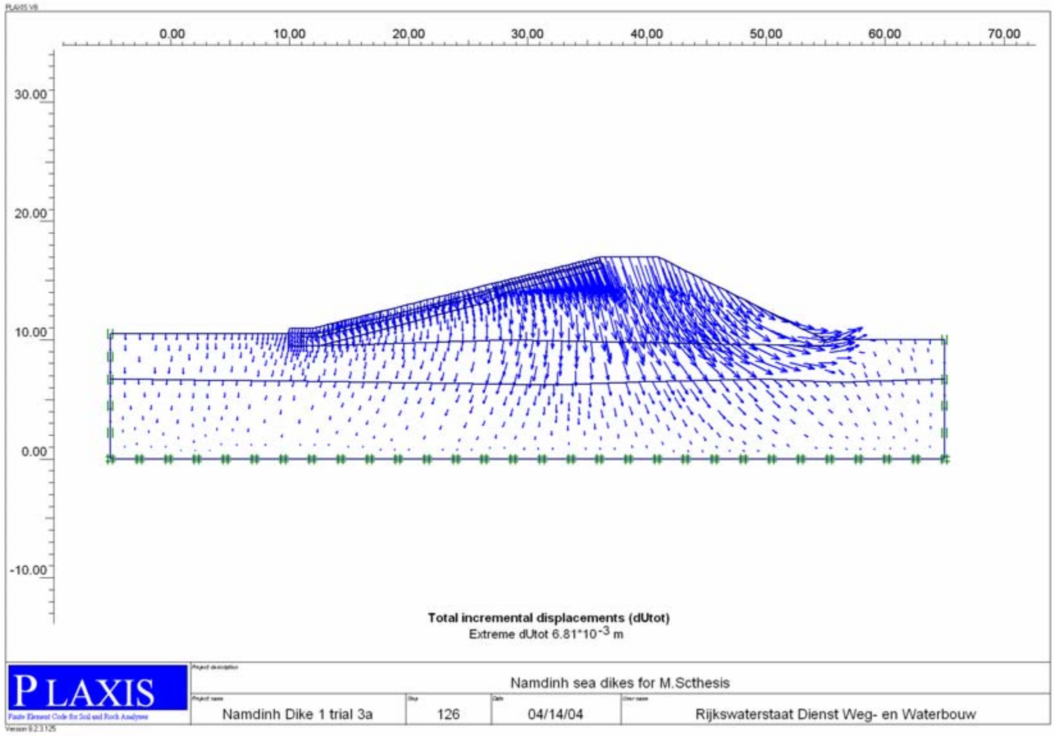
D : -60.000
E : -52.000
F : -44.000
G : -36.000
H : -28.000
I : -20.000
J : -12.000
K : -4.000
L : 4.000
Vertical displacements (Uy)
Extreme Uy -76.78*10^-3 m
PLAXIS
Namdinh sea dikes for M.Scthesis
Namdinh Dike 1 trial 3a
126
04/14/04
Rijkswaterstaat Dienst Weg- en Waterbouw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-10.00
Total incremental displacements (dUtot)
Extreme dUtot 6.81*10^-3 m
PLAXIS
Namdinh sea dikes for M.Scthesis
Namdinh Dike 1 trial 3a
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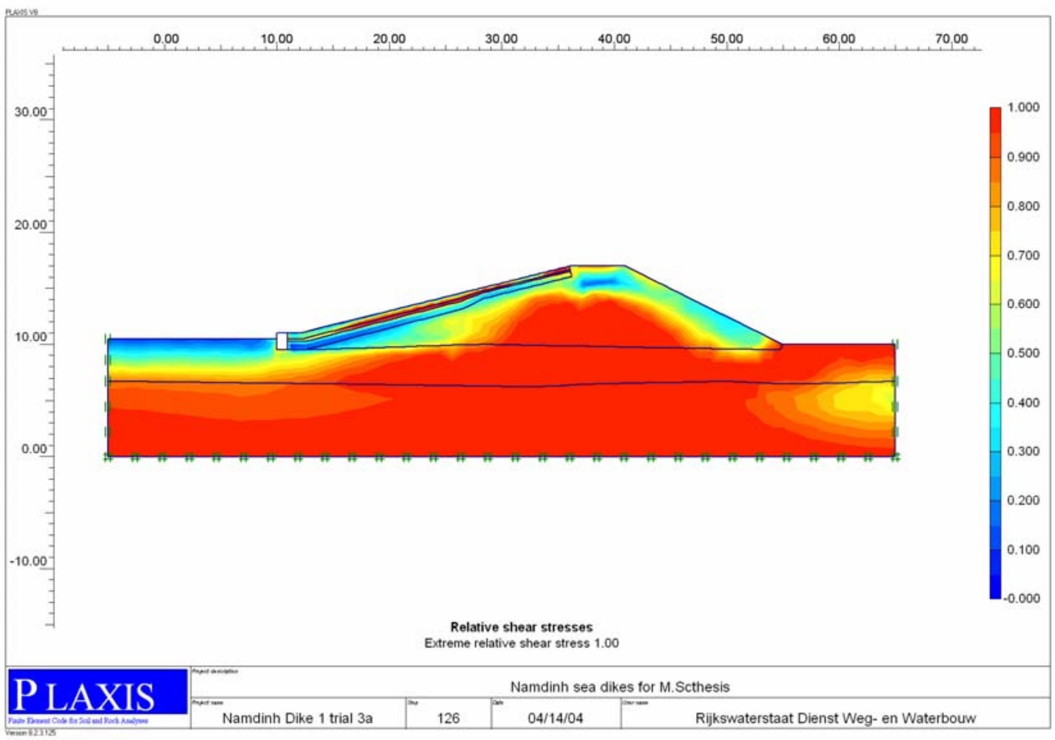
04/14/04
Rijkswaterstaat Dienst Weg- en Waterbouw


0,00
10,00
20,00
30,00
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60,00
70,00
30.00
20.00
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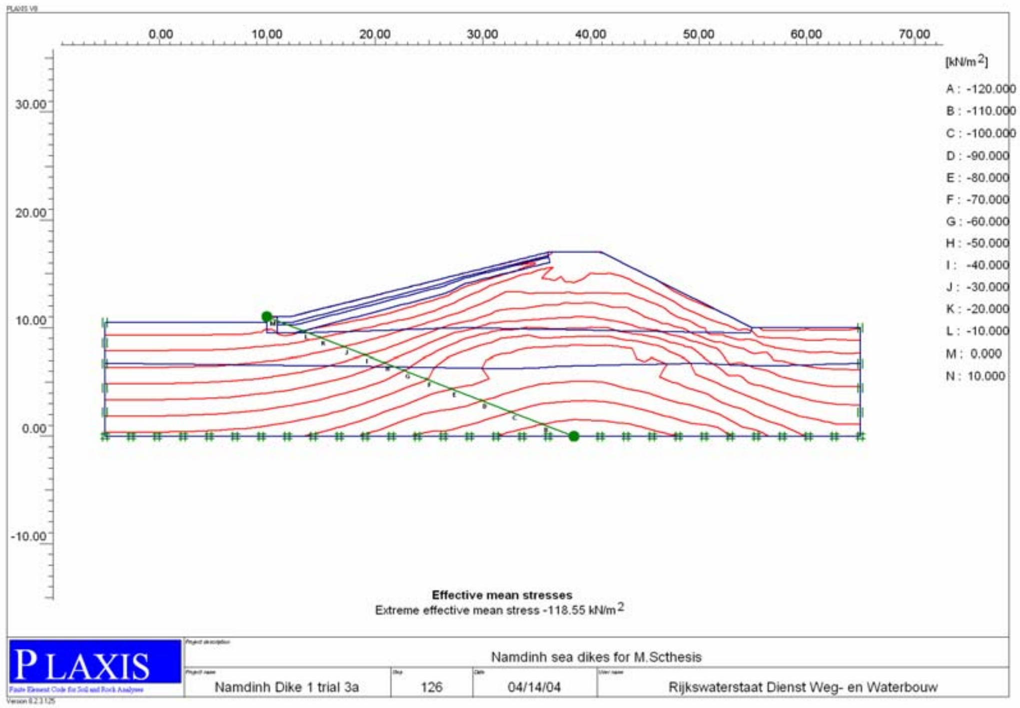
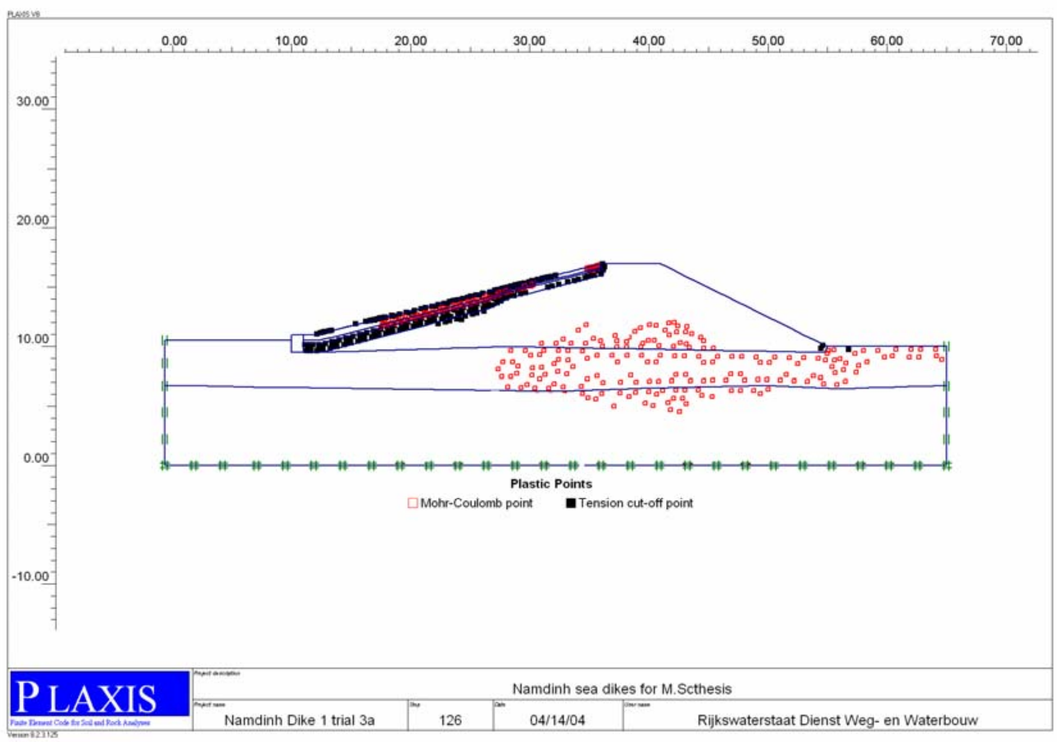
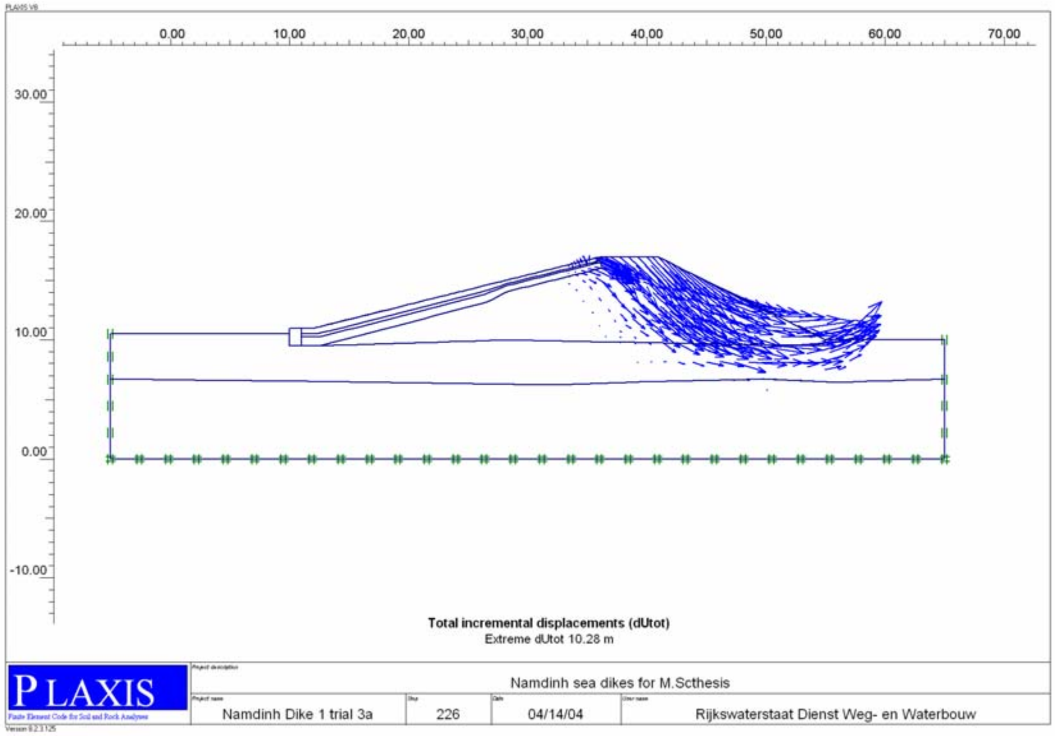
-10.00
1.000
0.900
0.800
0.700
0.600
0.500
0.400
0.300
0.200
0.100
-0.000
Relative shear stresses
Extreme relative shear stress 1.00
PLAXIS
Namdinh sea dikes for M.Scthesis
Namdinh Dike 1 trial 3a
126
04/14/04
Rijkswaterstaat Dienst Weg- en Waterbouw

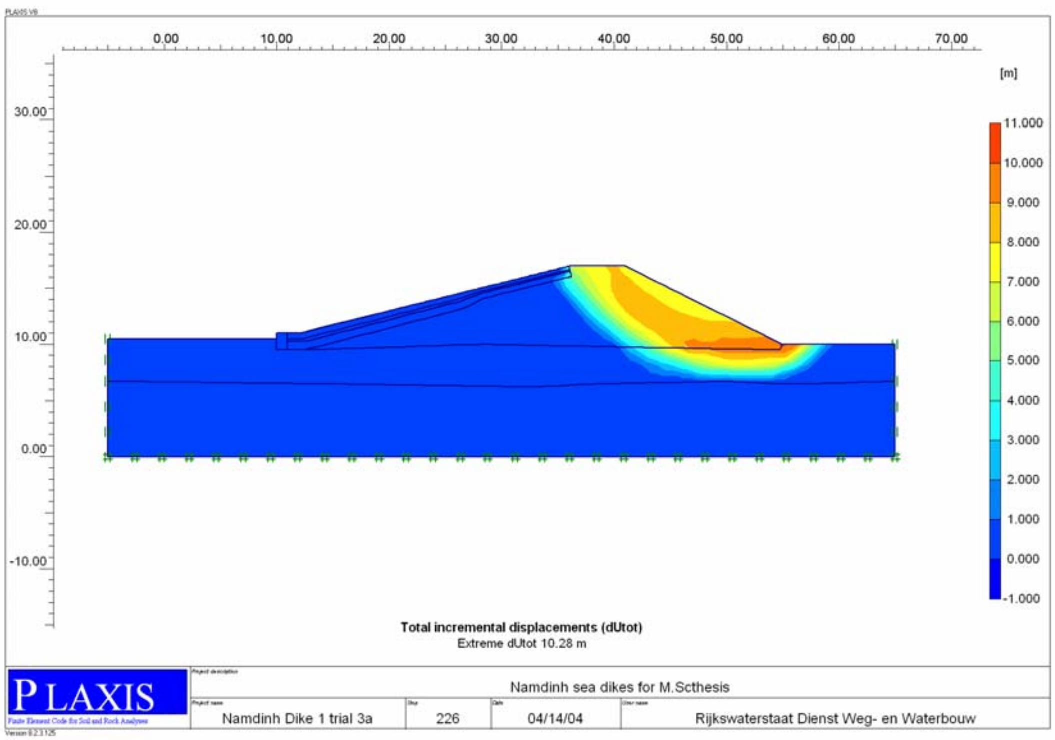


[kN/m2]
A : -120.000
B : -110.000
C : -100.000
D : -90.000
E : -80.000
F : -70.000
G : -60.000
H : -50.000
I : -40.000
J : -30.000
K : -20.000
L : -10.000
M : 0.000
N : 10.000
Effective mean stresses
Extreme effective mean stress -118.55 kN/m2
PLAXIS
Namdinh sea dikes for M.Scthesis
Namdinh Dike 1 trial 3a
126
04/14/04
Rijkswaterstaat Dienst Weg- en Waterbouw

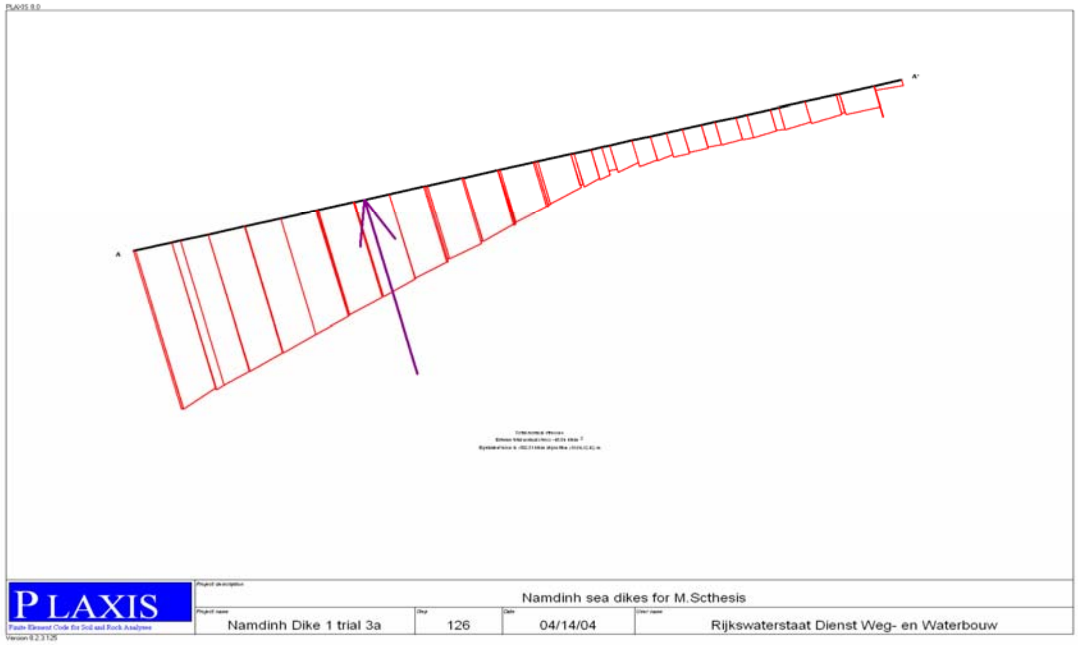


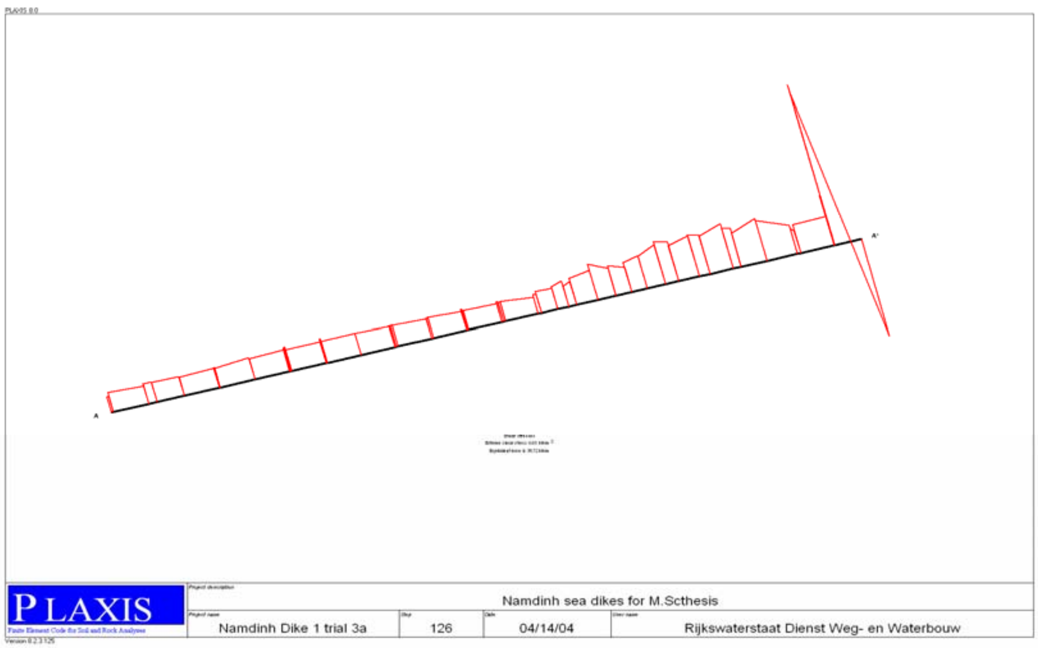
Plastic Points
Mohr-Coulomb point
Tension cut-off point
PLAXIS
Namdinh sea dikes for M.Scthesis
Namdinh Dike 1 trial 3a
126
04/14/04
Rijkswaterstaat Dienst Weg- en Waterbouw





Stresses on revetment


PLAXIS
Namdinh sea dikes for M.Scthesis
Namdinh Dike 1 trial 3a
126
04/14/04
Rijkswaterstaat Dienst Weg- en Waterbouw


PLAXIS
Namdinh sea dikes for M.Scthesis
Namdinh Dike 1 trial 3a
126
04/14/04
Rijkswaterstaat Dienst Weg- en Waterbouw

APPENDIX 2 USING GEO-SLOPE FOR CALCULATION OF SLOPE STABILITY.

2.1. Introduction

SLOPE/W is one module of GEO-Slope package. The software product that uses limit equilibrium theory to compute the factor of safety of earth and rock slopes. The comprehensive formulation of SLOPE/W makes it possible to easily analyze both simple and complex slope stability problems using a variety of methods to calculate the factor of safety.

2.2 Analysis Methods and theory

The comprehensive formulation of SLOPE/W allows stability analysis using the following methods:, Ordinary (or Fellenius) method, Bishop Simplified method, Janbu Simplified method, Spencer method, Morgenstern-Price method, Corps of Engineers method, Lowe-Karafiath method, generalized limit equilibrium (GLE) method, finite element stress method. Furthermore, a variety of interslice side force functions can be used with the more mathematically rigorous Morgenstern-Price and GLE methods.

This part explains the theory used in the development problem by using SLOPE/W. The variables used are first defined, followed by a brief description of the General Limit Equilibrium method (GLE). The relevant equations are derived, including the base normal force equation and the factor of safety equations. This is followed by a section describing the iterative procedure adopted in solving the nonlinear factor of safety equations. Attention is then given to aspects of the theory related to soils with negative pore-water pressures.

SLOPE/W solves two factor of safety equations; one satisfying force equilibrium and one satisfying moment equilibrium. All the commonly used methods of slices can be visualized as special cases of the General Limit Equilibrium (GLE) solution.

SLOPE/W uses the theory of limit equilibrium of forces and moments to compute the factor of safety against failure. The General Limit Equilibrium (GLE) theory is presented and used as the context for relating the factors of safety for all commonly used methods of slices.

A factor of safety is defined as that factor by which the shear strength of the soil must be reduced in order to bring the mass of soil into a state of limiting equilibrium along a selected slip surface.

For an effective stress analysis, the shear strength is defined as:

$$s = c' + (\sigma - u)\tan\varphi' \qquad (1)$$

where:
S = shear strength
c' = effective cohesion
ϕ' = effective angle of internal friction
σ_n = total normal stress
u = pore-water pressure

For a total stress analysis, the strength parameters are defined in terms of total stresses and pore-water pressures are not required.

The stability analysis involves passing a slip surface through the earth mass and dividing the inscribed portion into vertical slices. The slip surface may be circular, composite (i.e., combination of circular and linear portions) or consist of any shape defined by a series of straight lines (i.e., fully specified slip surface).

The limit equilibrium formulation assumes that:

1. The factor of safety of the cohesive component of strength and the frictional component of strength are equal for all soils involved.

2. The factor of safety is the same for all slices.

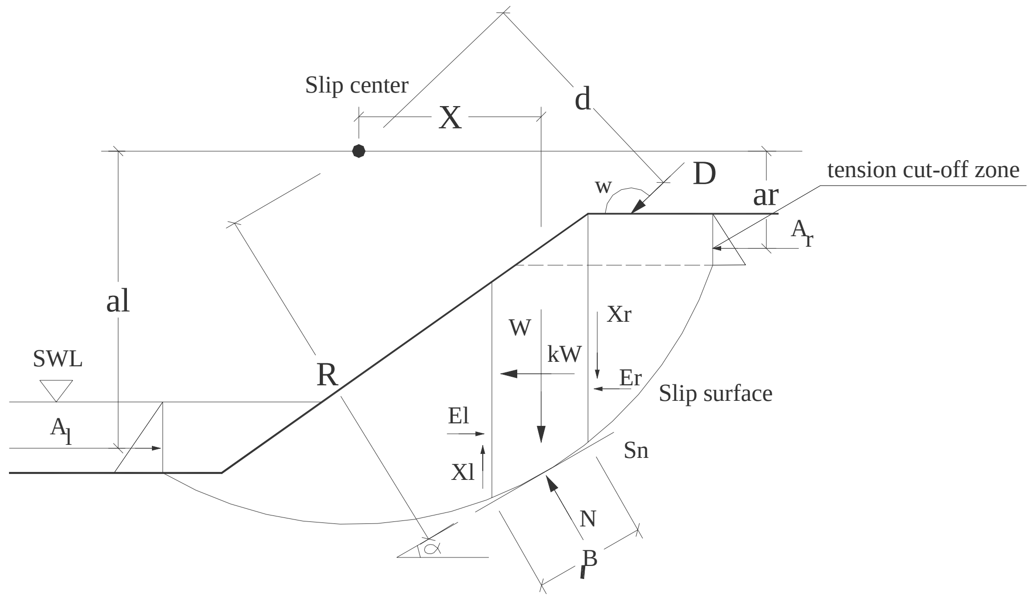


Figure 1: Forces Acting on a Slice Through a Sliding Mass with a Circular Slip Surface

Figures 1 shows all the forces acting on a circular and a composite slip surface. The variables are defined as follows:

W = the total weight of a slice of width b and height h
N = the total normal force on the base of the slice
S = the shear force mobilized on the base of each slice.
E = the horizontal interslice normal forces. Subscripts L and R designate the left and right sides of the slice, respectively.
X = the vertical interslice shear forces. Subscripts L and R define the left and right sides of the slice, respectively.
D = an external line load.
k_W = the horizontal seismic load applied through the centroid of each slice.
R = the radius for a circular slip surface or the moment arm associated with the mobilized shear force, S_m for any shape of slip surface.
f = the perpendicular offset of the normal force from the center of rotation or from the center of moments. It is assumed that f distances on the right side of the center of rotation of a negative slope (i.e., a right facing slope) are negative and those on the left side of the center of rotation are positive. For positive slopes, the sign convention is reversed.
x = the horizontal distance from the centerline of each slice to the center of rotation or to the center of moments.
e = the vertical distance from the centroid of each slice to the center of rotation or to the center of moments.
d = the perpendicular distance from a line load to the center of rotation or to the center of moments.
h = the vertical distance from the center of the base of each slice to the uppermost line in the geometry (i.e., generally ground surface).
a = the perpendicular distance from the resultant external water force to the center of rotation or to the center of moments. The L and R subscripts designate the left and right sides of the slope, respectively.
A = the resultant external water forces. The L and R subscripts designate the left and right sides of the slope, respectively.
ω = the angle of the line load from the horizontal. This angle is measured counter-clockwise from the positive x-axis.
α = the angle between the tangent to the center of the base of each slice and the horizontal. The sign convention is as follows. When the angle slopes in the same direction as the overall slope of the geometry, is positive, and vice versa.
u = Pore water pressure

+ Value of force S_m that satisfies critical equilibrium conditions is:

The magnitude of the shear force mobilized S_m to satisfy conditions of limiting equilibrium is:

$$S_m = \frac{S * \beta}{F} = \frac{\beta\left[c' + \left(\sigma_n - u\right)\tan\varphi\right]}{F} \qquad (2)$$

Where

$\sigma_n = \dfrac{N}{\beta}$ = average normal stress at the base of each slice

F = the factor of safety

β = the base length of each slice

The elements of statics that can be used to derive the factor of safety are the summations of forces in two directions and the summation of moments. These, along with failure criteria, are insufficient to make the problem determinate. More information must be known about either the normal force distribution at the base of the slices or the interslice force distribution. Since the number of unknown quantities exceeds the number of known quantities, the problem is indeterminate. Assumptions regarding the directions, magnitude, and/or point of application of some of the forces must be made to render the analysis determinate. Most methods first assume that the point of application of the normal force at the base of a slice acts through the centerline of the slice. Then an assumption is most commonly made concerning the magnitude, direction, or point of application of the interslice forces. In general, the various methods of slices can be classified in terms of (1) the statics used in deriving the factor of safety equation and (2) the interslice force assumption used to render the problem determinate.

3. General Limit Equilibrium Method

The General Limit Equilibrium method (GLE) uses the following equations of statics in solving for the factor of safety:

1. The summation of forces in a vertical direction for each slice. The equation is solved for the normal force at the base of the slice, N.

2. The summation of forces in a horizontal direction for each slice is used to compute the interslice normal force, E. This equation is applied in an integration manner across the sliding mass (i.e., from left to right).

3. The summation of moments about a common point for all slices. The equation can be rearranged and solved for the moment equilibrium factor of safety, F_m.

4. The summation of forces in a horizontal direction for all slices, giving rise to a force equilibrium factor of safety, F_f.

The analysis is still indeterminate, and a further assumption is made regarding the direction of the resultant interslice forces. The direction is assumed to be described by a interslice force function. The factors of safety can now be computed based on moment equilibrium (F_m) and force equilibrium (F_f). These factors of safety may vary depending on the percentage (λ) of the force function used in the computation.

The factor of safety satisfying both moment and force equilibrium is considered to be the converged factor of safety of the GLE method.

Reference can be made to Figures 1, for deriving the moment equilibrium factor of safety equation. In each case, the summation of moments for all slices about a common point, can be written as follows:

$$\sum W_X - \sum S_m R - \sum kW_e \pm [Dd] \pm Aa = 0 \qquad (3)$$

The term [Dd] in Equation 3 mean that these forces are considered only for the slice on which the forces act. Substituting Equation 2 into Equation 3 and solving for the factor of safety gives,

$$F_m = \frac{\sum\left(c'\beta R + (N - u\beta)R\tan\varphi'\right)}{\sum W_X - \sum Nf + \sum kW_e \pm (Dd) \pm Aa} \qquad (4)$$

Equation 4 is nonlinear since the normal force, N, is also a function of the factor of safety. The procedure for solving the equation is described in SLOPE/W Equations in this chapter.

Reference can be made to Figures 1 for deriving the force equilibrium factor of safety equation. The summation of forces in a horizontal direction for all slices gives,

$$\sum(E_L - E_R) - \sum(N \sin\alpha) + \sum(S_m \cos\alpha) - \sum(kW) \qquad (5)$$

The term $\sum(E_L - E_R) = 0$ must be zero when summed over the entire sliding mass. Substituting Equation 2 into Equation 5 and solving for the factor of safety gives,

$\sum(E_L - E_R) = 0$ total action forces at cylinder slip has to annul

Thus: $$F_f = \frac{\sum\left[c'\beta\cos\alpha + (N - u\beta)\tan\varphi'\cos\alpha\right]}{\sum N\sin\alpha + \sum kW - (D\cos\omega) \pm A} \qquad (6)$$

The normal force at the base of a slice is derived from the summation of forces in a vertical direction on each slice.

$$-W + (X_l - X_r) + N\cos\alpha + S_m \sin\alpha - [D\sin\omega] = 0 \qquad (7)$$

Substituting Equation 2 into 7 and solving for the normal force, *N*, gives,

$$N = \frac{W + (X_R - X_L) - \dfrac{c'\beta\sin\alpha + u\beta\sin\alpha\tan\varphi'}{F} + (D\sin\omega)}{\cos\alpha + \dfrac{\sin\alpha\tan\varphi'}{F}} \qquad (8)$$

The denominator in Equation 8 is commonly given the variable name, m_α. The factor of safety, F, is equal to the moment equilibrium factor of safety, F_m, when solving for moment equilibrium, and equal to the force factor of safety, F_f, when solving for force equilibrium.

Equation 8 cannot be solved directly since the factor of safety (F) and the interslice shear forces, (i.e., X_L and X_R) are unknown. The normal at the base of each slice is solved using an interactive scheme.

To commence the solution for the factor of safety, it is possible to neglect the interslice shear and normal forces on each slice (Fellenius, 1936). When forces are summed in a direction perpendicular to the base of each slice, the following equation is obtained for the normal force.

$$N = W\cos\alpha - kW\sin\alpha + \left[D\cos(\omega + \alpha - 90)\right] \qquad (9)$$

Using the simplified equation (Equation 9) in solving Equations 4 and 6 provides starting values for the factor of safety computations. The factor of safety from Equation 8.4 is the Fellenius or Ordinary method factor of safety.

Next, assuming that the interslice shear forces in Equation 8 are equal to zero, the normal force at the base can be computed by:

$$N = \frac{W - \dfrac{c'\beta\ \sin\ \alpha + u\beta\ \sin\ \alpha\ \tan\ \phi'}{F} + [D\ \sin\ \omega]}{\cos\ \alpha + \dfrac{\sin\ \alpha\ \tan\ \phi'}{F}} \qquad (10)$$

When Equation 10 is used in solving for the moment equilibrium factor of safety. The solution is the factor of safety for Bishop's Simplified method.

4. Result of calculation.

4.1 Stability analysis for outer slope.

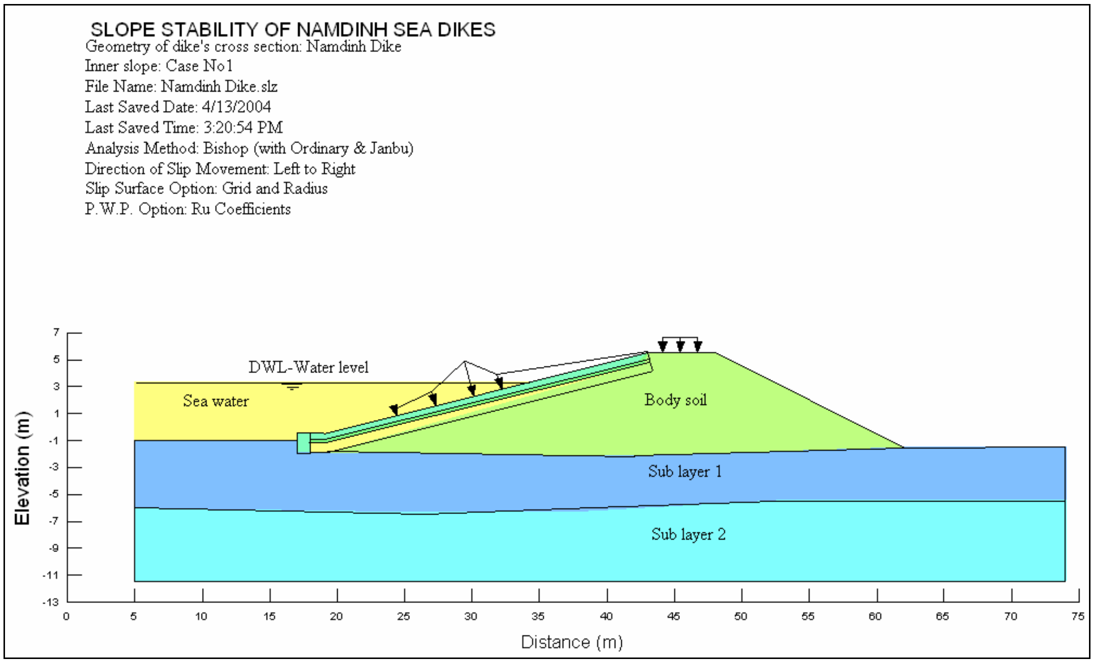


Figure 1: Geometry and load boundary condition at DWL

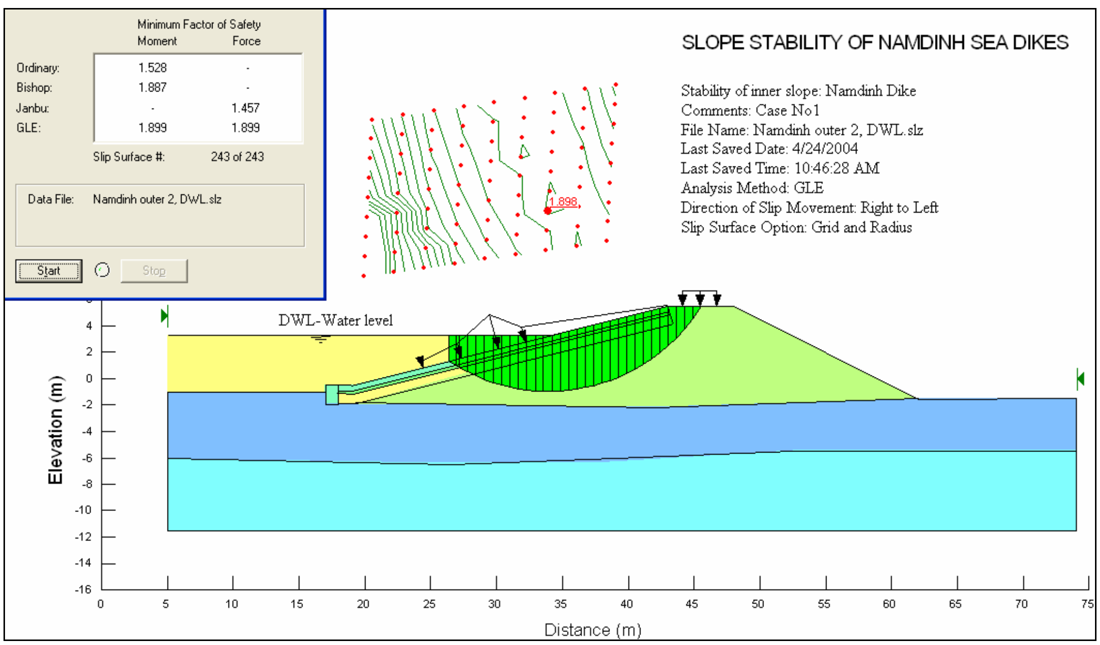


Figure 2: Stability of outer slope at DWL

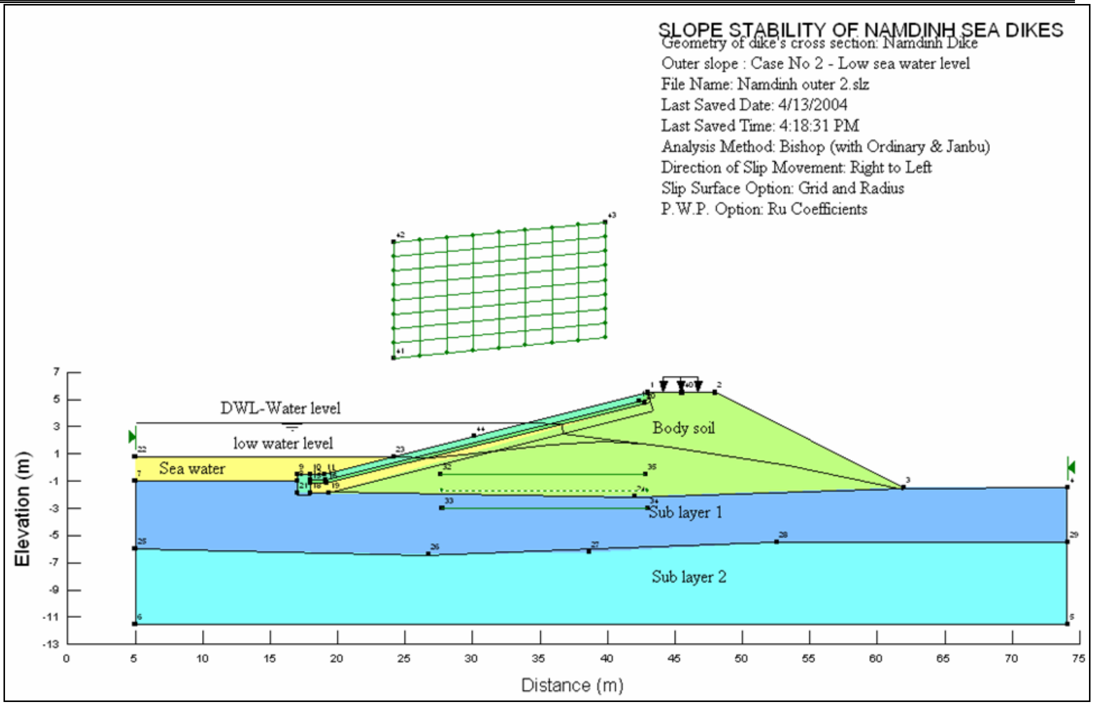


Figure 3: Geometry and load boundary condition at LWL

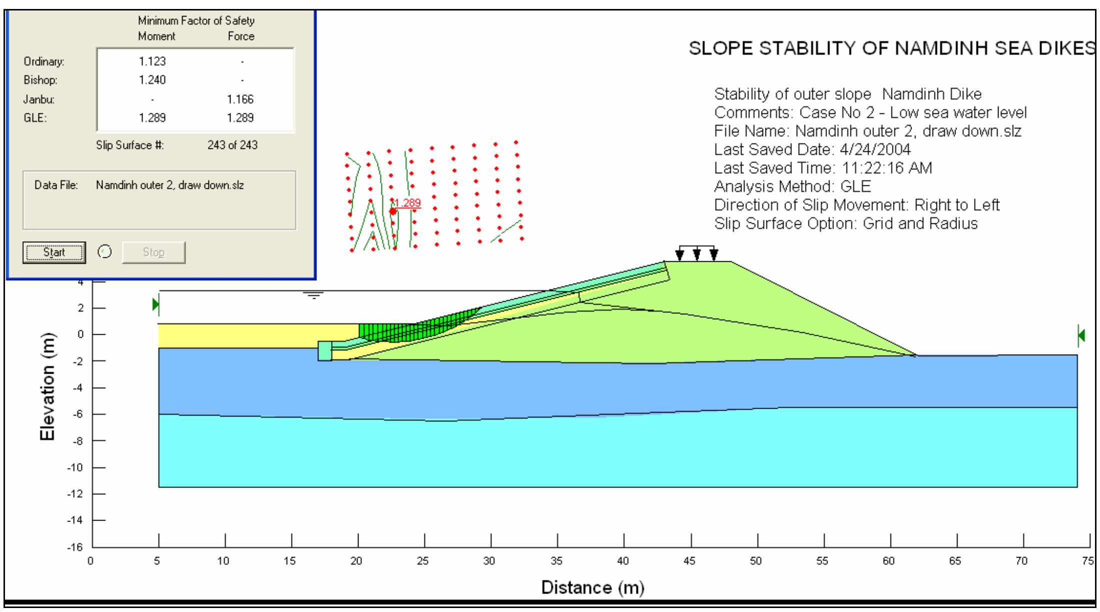


Figure 4: Stability of outer slope at LWL

4.2 Stability analysis for inner slope

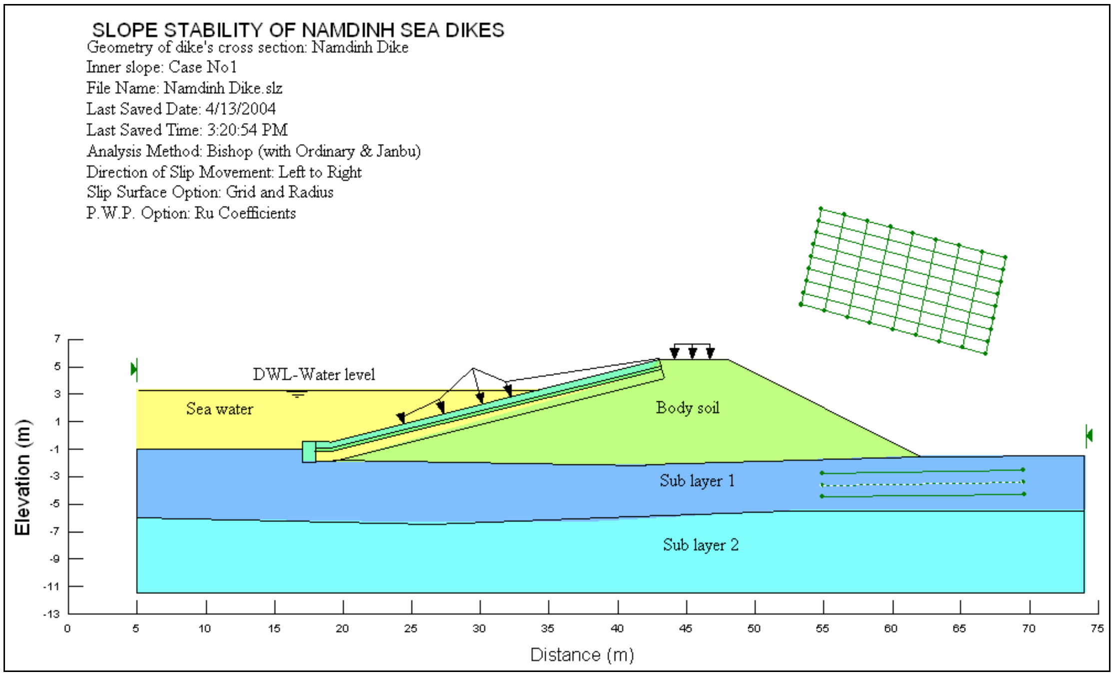


Figure 5: Geometry and load boundary condition at DWL

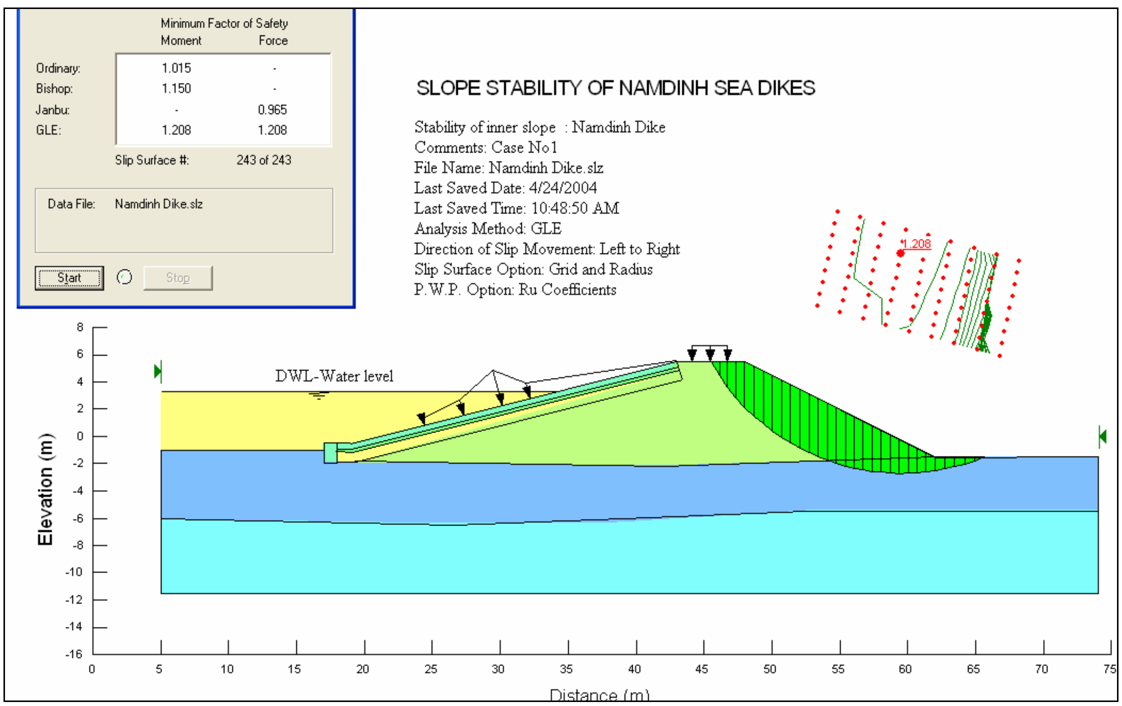


Figure 6: Stability of inner slope at DWL

APPENDIX 3 DATA AND RESULT BY USING VaP MODEL

APPENDIX 3 DATA AND RESUST BY USING VaP MODEL

OVTOP1A

OVERTOPPING - EXISTING DIKE-RIPRAP SLOPE PROTECTION

Limit State Function G :

```
G = Zc-MHWL-surge-SLRise-(K1*K2*K3*SQRT(a*((MHWL+Surge+SLRise)-Zbed-Zbed2)*
     Sqrt(9.81*((MHWL+Surge+SLRise)-Zbed))*Tm))/sqrt(1+slope^2)
```

Variables of G:

K1	N	0.550	0.050
K2	D	1.000	
K3	D	1.650	
MHWL	N	2.290	0.071
SLRise	N	0.000	0.050
Surge	N	1.000	0.200
Tm	D	8.500	
Zbed	D	-0.500	
Zbed2	N	0.000	0.200
Zc	N	5.500	0.200
a	N	0.500	0.050
slope	N	4.000	0.150

FORM Analysis of G:

HL - Index = 0.0646 P(G<0) = 0.474

Name	Alpha	Design Value
K1	0.447	0.551
MHWL	0.228	2.291
SLRise	0.161	5.199e-04
Surge	0.643	1.008
Zbed2	-0.130	-0.002
Zc	-0.449	5.494
a	0.246	0.501
slope	-0.174	3.998

Crude Monte Carlo Analysis of G:

1 run with 1000 samples:
1. m = 0.0452831 s = 0.440315 p = 0.453333

Numerical Integration (Evans) of G:

mean = 0.0302691 sdev = 0.445397 skew = -0.0756934 kurt = 2.99871

Respective Johnson curve
SB-variate: g = -8.20515, d = 12.487, xl = 24.7594, xi = -16.2708
ajv = 0: snv = -0.0804802 -> pf = 0.467928

Expectation of G:

E[G(X)] = 0.0287106

OVTOP1B

OVERTOPPING - EXISTING DIKE-PLACED BLOCK SLOPE PROTECTION

Limit State Function G :

```
G = Zc-MHWL-Surge-SLRise-(K1*K2*K3*SQRT(a*((MHWL+Surge+SLRise)-Zbed-Zbed2)*
   Sqrt(9.81*((MHWL+Surge+SLRise)-Zbed))*Tm))/sqrt(1+slope^2)
```

Variables of G:

K1	N	0.750	0.050
K2	D	1.000	
K3	D	1.650	
MHWL	N	2.290	0.071
SLRise	N	0.000	0.050
Surge	N	1.000	0.200
Tm	D	8.500	
Zbed	D	-0.500	
Zbed2	N	0.000	0.200
Zc	N	5.500	0.200
a	N	0.500	0.050
slope	N	4.000	0.150

FORM Analysis of G:

HL - Index = -0.338   P(G<0) = 0.632

Name	Alpha	Design Value
K1	0.402	0.743
MHWL	0.232	2.284
SLRise	0.163	-0.003
Surge	0.653	0.956
Zbed2	-0.160	0.011
Zc	-0.413	6.128
a	0.302	0.495
slope	-0.210	4.011

Crude Monte Carlo Analysis of G:

1 run with 1000 samples:
1.   m = -0.148673   s = 0.495227   p = 0.614444

Numerical Integration (Evans) of G:

mean = -0.16236 sdev = 0.48919 skew = -0.0805055 kurt = 2.99775

Respective Johnson curve
SB-variate: g = -7.3059, d = 11.4279, xl = 24.7545, xi = -16.36
ajv = 0: snv = 0.319415 -> pf = 0.625294

Expectation of G:

E[G(X)] = -0.164486

```
                                          OVETOP2A
OVERTOPPING - NEW DIKE BY DETERMINISTIC DESIGN, VIETNAMESE CODE
-RIPRAP SLOPE PROTECTION

     Limit State Function G :

        G = Zc-MHWL-Surge-SLRise-(K1*K2*K3*SQRT(a*((MHWL+Surge+SLRise)-Zbed-Zbed2)
                *Sqrt(9.81*((MHWL+Surge+SLRise)-Zbed))*Tm))/sqrt(1+slope^2)


 Variables of G:

     K1      N            0.550          0.050
     K2      D            1.000
     K3      D            1.650
     MHWL    N            2.290          0.071
     SLRise   N            0.000          0.050
     Surge   N            1.000          0.200
     Tm      D            8.500
     Zbed    D           -1.300
     Zbed2   N            0.000          0.200
     Zc      N            6.600          0.200
     a       N            0.500          0.050
     slope   N            4.000          0.150
      FORM Analysis of G:

     HL - Index = 1.67         P(G<0) =  0.0474

     Name        Alpha        Design Value
     K1          0.510           0.593
     MHWL        0.212           2.315
     SLRise        0.150           0.012
     Surge        0.598           1.200
     Zbed2       -0.124          -0.041
     Zc         -0.412          6.462
     a           0.288          0.524
     slope       -0.216           3.946


 Crude Monte Carlo Analysis of G:

     1 run with 1000 samples:
     1.   m = 0.800482     s = 0.484358     p = 0.0588889


 Numerical Integration (Evans) of G:

     mean = 0.793049  sdev = 0.463078  skew = -0.0838493  kurt = 2.999

     Respective Johnson curve
     SB-variate: g = -7.71108, d = 11.486, xl = 23.7906, xi = -14.9451
     ajv =  0: snv = -1.68683 -> pf = 0.0458179


 Expectation of G:

     E[G(X)] = 0.791782
```


E[G(X)] = 0.791782

```
                                             OVETOP2B
OVERTOPPING - NEW DIKE BY DETERMINISTIC DESIGN, VIETNAMESE CODE
 PLACED BLOCK SLOPE PROTECTION

   Limit State Function G :

     G = Zc-MHWL-Surge-SLRise-(K1*K2*K3*SQRT(a*((MHWL+Surge+SLRise)-Zbed-Zbed2)*
        Sqrt(9.81*((MHWL+Surge+SLRise)-Zbed))*Tm))/sqrt(1+slope^2)


 Variables of G:

     K1      N            0.750          0.050
     K2      D            1.000
     K3      D            1.650
     MHWL    N            2.290          0.071
     SLRise   N            0.000          0.050
     Surge   N            1.000          0.200
     Tm      D            8.500
     Zbed    D           -1.300
     Zbed2   N            0.000          0.200
     Zc      N            7.600          0.200
     a       N            0.500          0.050
     slope   N            4.000          0.150
    FORM Analysis of G:

     HL - Index = 1.68         P(G<0) =  0.0464

     Name        Alpha        Design Value
     K1          0.467           0.789
     MHWL        0.213           2.315
     SLRise        0.150           0.013
     Surge        0.601           1.202
     Zbed2       -0.151          -0.051
     Zc         -0.374           7.474
     a           0.348           0.529
     slope       -0.264           3.933


 Crude Monte Carlo Analysis of G:

     1 run with 1000 samples:
     1.   m = 0.87193     s = 0.501313     p = 0.0455556


 Numerical Integration (Evans) of G:

     mean = 0.877794  sdev = 0.508773  skew = -0.0894245  kurt = 2.99775

     Respective Johnson curve
     SB-variate: g = -6.7796, d = 10.4312, xl = 23.583, xi = -14.6085
     ajv =  0: snv = -1.69738 -> pf = 0.0448123


 Expectation of G:

     E[G(X)] = 0.876066
```

OVERTOPPING - New DIKE-Riprap SLOPE PROTECTION- New Dutch code

Limit State Function G1 :

```
G1 = Zc-MHWL-Surge-SLRise-model*K1*K2*K3*a*
(((MHWL+Surge+SLRise)-Zbed-Zbed2))/
Sqrt((2*pi*a*((MHWL+Surge+SLRise)-Zbed)/9.81/Top^2))/slope
```

Variables of G1:

K1	N	0.550	0.050
K2	D	1.000	
K3	D	0.950	
MHWL	N	2.290	0.071
SLRise	N	0.100	0.050
Surge	N	1.000	0.200
Top	D	10.200	
Zbed	D	-1.300	
Zbed2	N	0.000	0.200
Zc	N	8.750	0.200
a	N	0.500	0.050
model	N	1.650	0.116
slope	N	4.000	0.150

FORM Analysis of G1:

HL - Index = 1.64 $\quad$ P(G<0) = 0.0501

Name	Alpha	Design Value
K1	0.523	0.593
MHWL	0.159	2.309
SLRise	0.112	0.109
Surge	0.449	1.147
Zbed2	-0.272	-0.090
Zc	-0.315	7.496
a	0.295	0.524
model	0.414	1.729
slope	-0.236	3.942

Over3B

OVERTOPPING - New DIKE-Block SLOPE PROTECTION- New Dutch code

Limit State Function G2 :

```
    G2 =
Zc-MHWL-Surge-SLRise-model*K1*K2*K3b*a*(((MHWL+Surge+SLRise)-Zbed-Zbed2))
                /Sqrt((2*pi*a*((MHWL+Surge+SLRise)-Zbed)/9.81/Top^2))/slope
```

Variables of G2:

K1	N	0.750	0.050
K2	D	1.000	
K3b	D	0.95	
MHWL	N	2.290	0.071
SLRise	N	0.100	0.050
Surge	N	1.000	0.200
Top	D	10.200	
Zbed	D	-1.300	
Zbed2	N	0.000	0.200
Zc	N	8.750	0.200
a	N	0.500	0.050
model	N	1.650	0.116
slope	N	4.000	0.150

FORM Analysis of G2:

HL - Index = 2.05 P(G<0) = 0.0201

Name	Alpha	Design Value
K1	0.453	0.796
MHWL	0.152	2.312
SLRise	0.107	0.111
Surge	0.427	1.175
Zbed2	-0.288	-0.118
Zc	-0.286	8.633
a	0.337	0.535
model	0.473	1.762
slope	-0.276	3.915

APPENDIX 3 DATA AND RESUST BY USING VaP MODEL

```
                                   ARMPIR1
INSTABILITY OF ARMOUR LAYER OF PLACED BLOCK SLOPE PROTECTION
FOR EXISTING DIKE

Limit State Function G :

    G = Phi*Del*d-Hs*(slope/SQRT(sm))^(b)/cosina


Variables of G:

    Del     N            1.400          0.050
    Hs      LN           0.634          0.128
    Phi     N            5.000          0.500
    b       N            0.650          0.150
    cosina   D            0.970
    d       D            0.500
    slope   N            0.250          0.018
    sm      D            0.020


FORM Analysis of G:

    HL - Index = 1.11         P(G<0) =   0.132

    Name         Alpha        Design Value
    Del         -0.181           1.390
    Hs           0.646           2.067
    Phi         -0.535           4.702
    b            0.445           0.724
    slope         0.257           0.255


Crude Monte Carlo Analysis of G:

    1 run with 1000 samples:
    1.   m = 0.634596     s = 0.603782     p = 0.143333


Numerical Integration (Evans) of G:

    mean = 0.654447  sdev = 0.595004  skew = -0.241836  kurt = 3.10482

    Respective Johnson curve
    SL-variate: g = -24.8771, d = 12.4519, xl = -1, xi = 8.05145
    ajv =  0: snv = -1.0958 -> pf = 0.136584


Expectation of G:

    E[G(X)] = 0.663335
```

```
                                    ARMPIR2
  INSTABILITY OF ARMOUR LAYER OF PLACED BLOCK SLOPE PROTECTION
  FOR NEW DIKE DESIGN BY DETERMINISTIC APPROACH IN VIETNAMESE CODE,
    Limit State Function G :

    G = Phi*Del*d-Hs*(slope/SQRT(sm))^(b)/cosina


Variables of G:

    Del     N            1.400          0.050
    Hs      LN           0.826          0.117
    Phi     N            5.000          0.500
    b       N            0.650          0.150
    cosina   D            0.970
    d       D            0.750
    slope   N            0.250          0.018
    sm      D            0.020


FORM Analysis of G:

    HL - Index = 2.25         P(G<0) =  0.0123

    Name         Alpha        Design Value
    Del         -0.182           1.380
    Hs           0.585           2.663
    Phi         -0.577           4.352
    b            0.463           0.806
    slope         0.279           0.261


Crude Monte Carlo Analysis of G:

    1 run with 1000 samples:
    1.   m = 1.79109     s = 0.783554     p = 0.0166667


Numerical Integration (Evans) of G:

    mean = 1.80538  sdev = 0.769396  skew = -0.146588  kurt = 3.04151

    Respective Johnson curve
    SL-variate: g = -56.4853, d = 20.4939, xl = -1, xi = 17.564
    ajv =  0: snv = -2.24726 -> pf = 0.0123117


Expectation of G:

    E[G(X)] = 1.81614
```

APPENDIX 3 DATA AND RESUST BY USING VaP MODEL

```
                                          ARMPIR3
   INSTABILITY OF ARMOUR LAYER OF PLACED BLOCK SLOPE PROTECTION
  FOR NEW DIKE DESIGN BY DETERMINISTIC APPROACH IN DUTCH CODE,

      Limit State Function G :

    G = Phi*Del*d-Hs*(slope/SQRT(sm))^(b)/cosina


Variables of G:

    Del     N            1.400          0.050
    Hs      LN           0.826          0.117
    Phi     N            5.000          0.500
    b       N            0.650          0.150
    cosina   D            0.970
    d       D            0.700
    slope   N            0.250          0.018
    sm      D            0.020


FORM Analysis of G:

    HL - Index = 1.9          P(G<0) =  0.0288

    Name        Alpha        Design Value
    Del        -0.184           1.383
    Hs          0.592           2.605
    Phi        -0.570           4.459
    b           0.463           0.782
    slope        0.276           0.259


Crude Monte Carlo Analysis of G:

    1 run with 1000 samples:
    1.   m = 1.4591     s = 0.773342     p = 0.0377778


Numerical Integration (Evans) of G:

    mean = 1.45538  sdev = 0.742878  skew = -0.167974  kurt = 3.05184

    Respective Johnson curve
    SL-variate: g = -46.2488, d = 17.8925, xl = -1, xi = 14.7369
    ajv =  0: snv = -1.88842 -> pf = 0.0294848


Expectation of G:

    E[G(X)] = 1.46614
```

APPENDIX 3 DATA AND RESUST BY USING VaP MODEL

```
 INSTABILITY OF ARMOUR LAYER OF SLOPE PROTECTION
 FOR EXISTING DIKE, RIPRAP REVETMENT
   Limit State Function G :

     G = model*(P^0.18)*(S/SQRT(N))^0.2*(slope/SQRT(sm))^(-0.5)-Hs/
(del*d)

 Variables of G:

     Hs      LN          0.634          0.128
     N       D       7.000e+03
     P       N           0.150          0.030
     S       D           2.000
     d       D           0.450
     del     N           1.600          0.050
     model   N           8.700          0.566
     slope   N           0.250          0.018
     sm      D           0.020

 FORM Analysis of G:

     HL - Index = 0.0671       P(G<0) =   0.473

     Name        Alpha        Design Value
     Hs           0.824          1.898
     P           -0.231          0.150
     del         -0.200          1.599
     model        -0.418          8.684
     slope         0.230          0.250
```

```
  INSTABILITY OF ARMOUR LAYER OF SLOPE PROTECTION
  FOR NEW DIKE DESIGN BY DETERMINISTIC APPROACH IN VIETNAMESE CODE,
   RIPRAP REVETMENT

     Limit State Function G :

     G = model*(P^0.18)*(S/SQRT(N))^0.2*(slope/SQRT(sm))^(-0.5)-Hs/
(del*d)


 Variables of G:

     Hs      LN           0.826          0.117
     N       D        7.000e+03
     P       N            0.150          0.030
     S       D            2.000
     d       D            0.890
     del     N            1.600          0.050
     model   N            8.700          0.566
     slope   N            0.250          0.018
     sm      D            0.020


 FORM Analysis of G:

     HL - Index = 2.15         P(G<0) =  0.0157

     Name        Alpha        Design Value
     Hs           0.781          2.778
     P           -0.273          0.132
     del         -0.212          1.577
     model        -0.466          8.133
     slope         0.233          0.259
```

APPENDIX 3 DATA AND RESUST BY USING VaP MODEL

```
   INSTABILITY OF ARMOUR LAYER OF SLOPE PROTECTION
   FOR NEW DIKE DESIGN BY DETERMINISTIC APPROACH IN DUTCH CODE,
   RIPRAP REVETMENT  Limit State Function G :

       Limit State Function G :

     G = model*(P^0.18)*(S/SQRT(N))^0.2*(slope/SQRT(sm))^(-0.5)-Hs/
(del*d)


 Variables of G:

     Hs      LN          0.826          0.117
     N       D       7.000e+03
     P       N           0.150          0.030
     S       D           2.000
     d       D           0.860
     del     N           1.600          0.050
     model   N           8.700          0.566
     slope   N           0.250          0.018
     sm      D           0.020


 FORM Analysis of G:

     HL - Index = 1.92         P(G<0) =  0.0274

     Name        Alpha        Design Value
     Hs           0.783          2.722
     P           -0.269          0.134
     del         -0.212          1.580
     model        -0.463          8.196
     slope         0.234          0.258
```

PIPING1&2

```
FAILURE DUE TO CONDITION 1 OF PIPING
  Limit State Function G1 :

     G1 = desic*g*d-desiw*g*(MHWL+Surge-Zinland)

  FORM Analysis of G1:

     HL - Index = 6.72         P(G<0) = 9.03e-12

     Name        Alpha        Design Value
     MHWL        0.169           2.371
     Surge        0.477           1.641
     Zinland        -0.596        -1.001e+00
     d          -0.624           2.871


FAILURE DUE TO CONDITION 2 OF PIPING
  Limit State Function G2 :

     G2 =m*(Lt+d)/c-(MHWL+Surge-Zinland)


 Variables of G2:

     Lt      N           46.000          5.000
     MHWL    N            2.290          0.071
     Surge   N            1.000          0.200
     Zinland   N          0.000          0.250
     c       D           15.000
     d       N            3.500          0.600
     m       N            1.670          0.330


   FORM Analysis of G2:

     HL - Index = 3.21         P(G<0) = 0.000657

     Name        Alpha        Design Value
     Lt         -0.783          33.421
     MHWL        0.108           2.315
     Surge        0.303           1.195
     Zinland        -0.379          -0.304
     d          -0.031           3.480
     m          -0.373           1.550
```

Establish Depth Limited Wave Height Distribution
****

For deisgn situation
Limit State Function G1 :
G1 = a*(MHWL+Surge-Zbed-Zbed2)

Variables of G:

MHWL	N	2.290	0.071
Surge	N	1.000	0.200
Zbed	N	0.000	0.200
Zbed2	D	-0.500	
a	N	0.500	0.050

Crude Monte Carlo Analysis of G1:

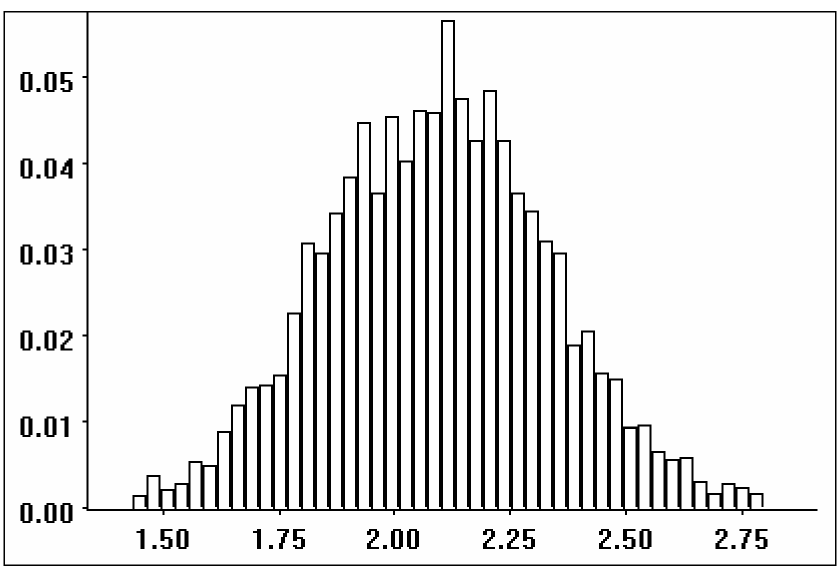
1 run with 1000 samples:
1. m = 1.88553 s = 0.245799 p = 0

Crude Monte Carlo Analysis of G1:

1 run with 3000 samples:
1. m = 1.90341 s = 0.243011 p = 0

Crude Monte Carlo Analysis of G1:

1 run with 2000 samples:
1. m = 1.90259 s = 0.230482 p = 0



Establish Depth Limited Wave Height Distribution
****
For Present situation

Limit State Function G2 :
G2 = a*(MHWL+Surge-Zbed-Zbed3)

Variables of G2:

MHWL	N	2.290	0.071
Surge	N	1.000	0.200
Zbed	N	0.000	0.200
Zbed3	D	-1.300	
a	N	0.500	0.050

Crude Monte Carlo Analysis of G2:

1 run with 1000 samples:
1. m = 2.30192 s = 0.271393 p = 0

Crude Monte Carlo Analysis of G2:

1 run with 2000 samples:
1. m = 2.29719 s = 0.279582 p = 0

Crude Monte Carlo Analysis of G2:
1 run with 3000 samples:
1. m = 2.27761 s = 0.266998 p = 0